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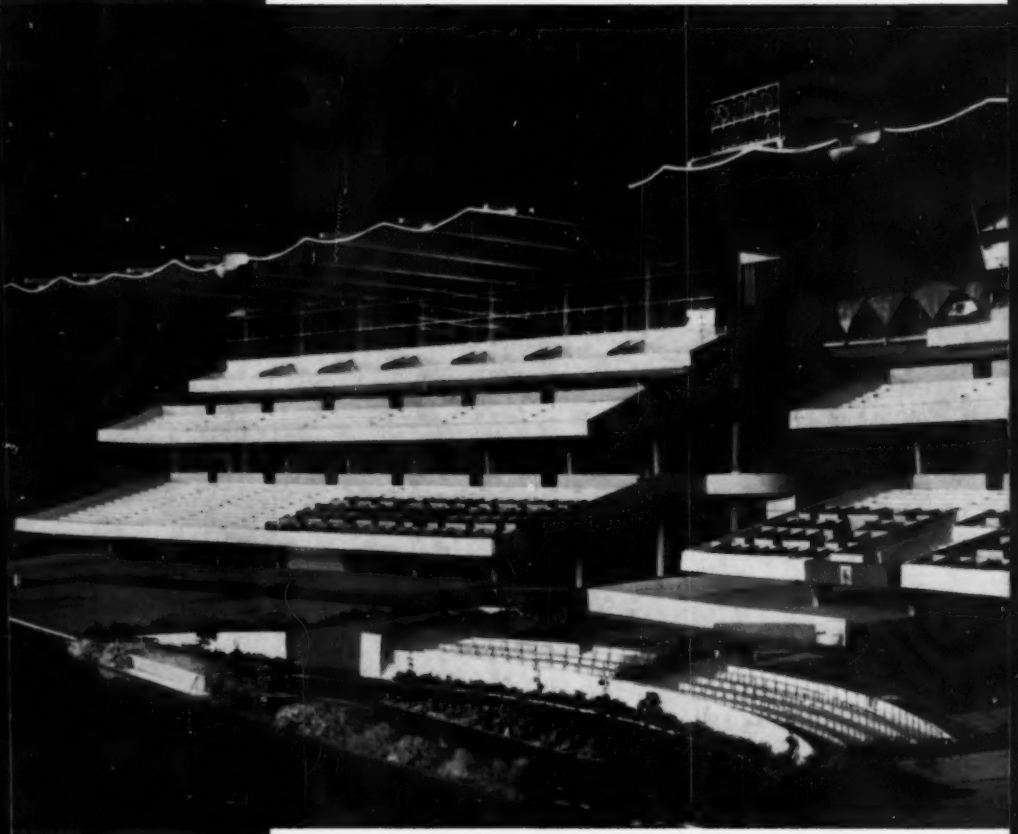
NOVEMBER 1959
No. 5
V. 31

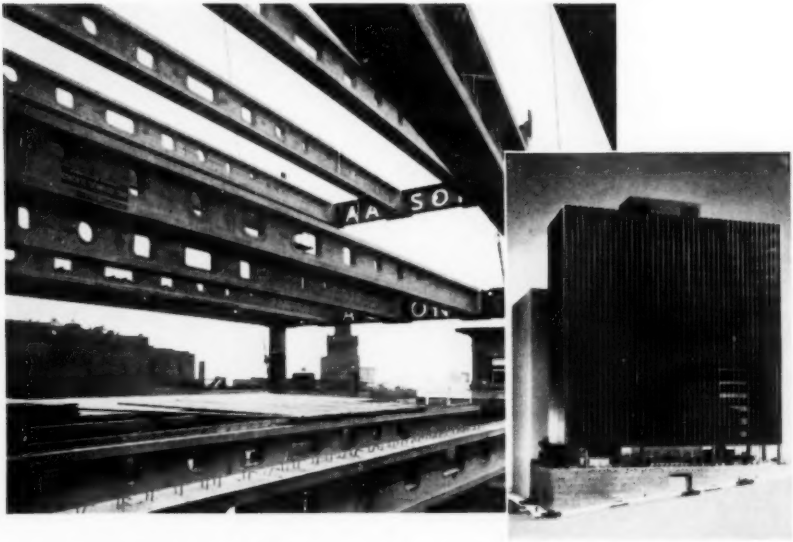
JOURNAL
OF THE
AMERICAN CONCRETE INSTITUTE



Report of the Committee on Nominations for 1960 appears on p. 1 of News Letter

ACI
PROCEEDINGS
V. 56

New Heights for P/S Concrete



• *Architects:* Bindon & Wright, Seattle • *Consultants:* Anderson, Birkeland & Anderson; Skidmore, Owings & Merrill; T. Y. Lin • *General Contractor:* Howard S. Wright Const. Co., Seattle • *P S Beams:* Concrete Technology Corp., Tacoma • *Ready-Mix Concrete:* Pioneer Sand & Gravel Co., Seattle

● Prestressed concrete has attained new heights in Seattle's 21-story Norton Building, first U.S. structure over six floors to utilize P/S. The unique design uses 238 P/S beams in combination with a steel perimeter frame to support column-free floors in the 17-story tower. Each 15-ton beam spans 70′, is only 37″ deep.

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Advertising. See statement under "Alphabetical List of Advertisers," final page **News Letter section.**

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Bound volumes 1 to 54 of PROCEEDINGS of the AMERICAN CONCRETE INSTITUTE (1905 to 1958) are for sale as far as available at prices to be had on inquiry of the Secretary-Treasurer. Special prices apply for members ordering bound volumes in addition to the monthly Journal.

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Publication office: P.O. Box 324, Shepherdsville, Ky. Entered in the Post Office at Shepherdsville, Ky., as mail of the second class.

Editorial and advertising departments at ACI headquarters, P.O. Box 4754, Redford Station, Detroit 19, Mich.

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Published by the American Concrete Institute under the authority of its Board of Direction

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"Behavior of a Continuous Concrete Slab Prestressed in Two Directions" is discussed by A. C. SCORDELIS, T. Y. LIN, and R. ITAYA. Paper is based on recent research by the author-trio on elastic behavior and ultimate strength of a continuous concrete slab prestressed in two directions.

WILLIAM L. ZEMAITIS reports on "An Investigation of Some Factors Affecting the Performance of Mortar for Unit Masonry" in an effort to provide a better understanding of these mortar failures and to determine corrective measures.

"Self-Service Parking Structures" by RICHARD C. RICH and WILLIAM J. ROUKE describes a 630-car facility completed last year in Edmonton, Alberta.

L. S. MULLER proposes a method for the "Design of Corner-Columns of L-Section Symmetrical for a 45-deg Axis with Small Eccentricities in any Direction."

B. KROONE and F. A. BLAKEY report on "Some Investigations of the Reaction Between Carbon Dioxide Gas and Portland Cement Mortar." Authors discuss the influence of such factors as evaporable and nonevaporable water as well as the effect of different storage conditions.

HOWARD L. FURR presents "A Numerical Method for an Approximate Analysis of Building Slabs" and discusses research involved in developing this method.

Discussion of papers published in the April, May, and June, 1959 issues will appear in the December JOURNAL in accordance with the Institute's quarterly publication schedule for *Proceedings* discussion.

On the Cover—The new Hipodromo Nacional (National Race Track) now under construction in Caracas, Venezuela. A paper based on the prestressed concrete thin shell cantilever roofs over the three large five-story grandstands appears on p. 409.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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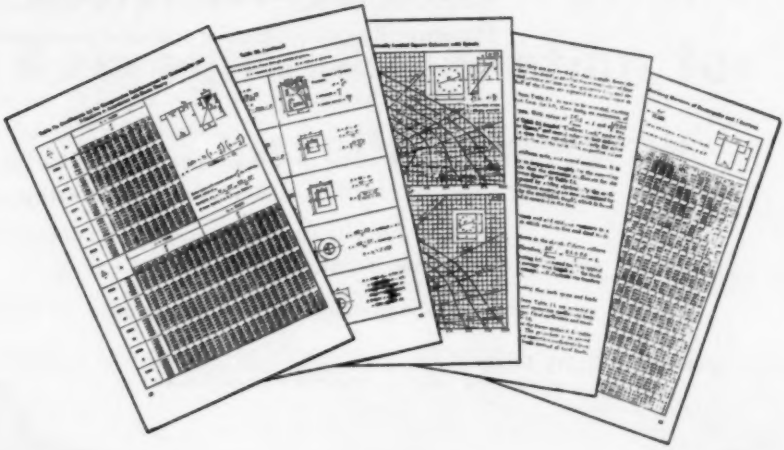

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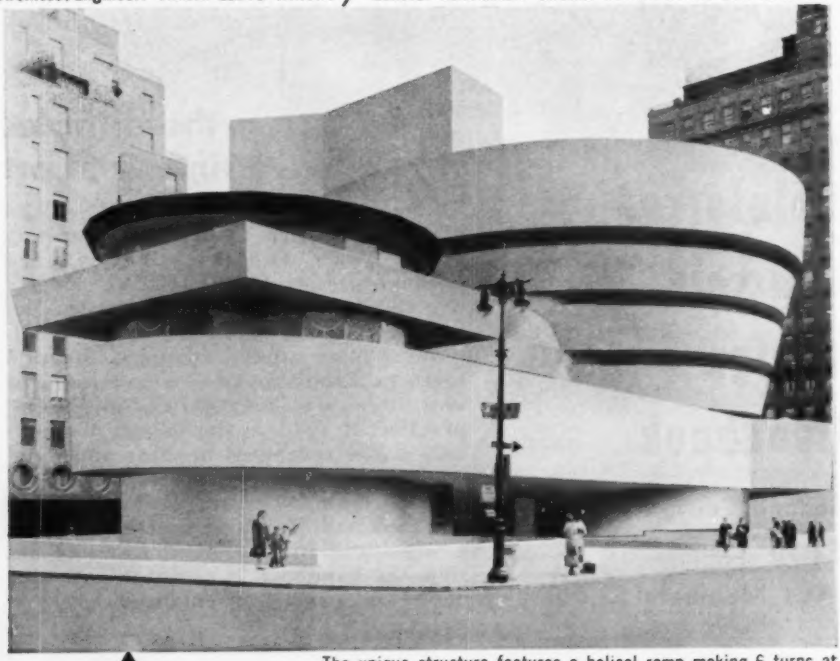

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Concrete Technology and Aggregate Production for St. Lawrence Seaway

By M. R. SMITH and GORDON M. KIDD

Curing practices and high lift versus low lift construction were sources of the major differences between Canadian and United States practices on the St. Lawrence Seaway. Contrasting materials and mixing specification requirements are also reported. Both United States and Canadian aggregate production procedures and equipment are detailed.

THE COMPLETED ST. LAWRENCE SEAWAY, officially opened in the spring of 1959, provides a waterway for world shipping from the mouth of the St. Lawrence River to the head of the Great Lakes, a distance of 2250 miles. The seaway accommodates shipping cargoes up to 25,000 tons in vessels having a length of 715 ft, a beam of 72 ft and a draft of 25 ft when fully loaded, to traverse an unimpeded waterway of 2013 miles.

Taking advantage of all the natural features of the St. Lawrence River and interconnecting lake channels, this waterway completes the plan started with the building of the Welland Canal from Lake Ontario to Lake Erie, finished in August, 1932. The locks and channels in the present construction are duplicates to a large degree of the Welland Canal system.

The recently completed construction for the seaway between Kingston and Montreal, consists of seven locks, two built by the United States and five by Canada. These seven locks and the seven Welland Canal locks establish the limits of seaway navigation, having the same over-all dimensions: 800 ft long, 80 ft wide, and 30 ft deep over the sills. The St. Lawrence Seaway Authority of Canada was in charge of the Canadian portion of construction. The U. S. Army Corps of Engineers, acting for the St. Lawrence Seaway Development Corp. (United States), was responsible for construction undertaken by the United States.

ACI member M. R. SMITH is assistant chief of the concrete branch, civil works, Office of the Chief of Engineers, Corps of Engineers, U. S. Army, Washington, D. C. Following experience as concrete control engineer for Garrison and Oahe dams on the Missouri River, his work on the St. Lawrence Seaway covered all phases of concrete construction. One of his major responsibilities was supervision of concrete aggregate production for the Corps of Engineers seaway projects. Mr. Smith recently became chairman of ACI Committee 611, Inspection of Concrete, and he is active in the technical committee work of ASTM.

GORDON M. KIDD, sales engineer with Construction Chemicals, Ltd., Toronto, Ontario, is also a member of the Institute. Before assuming his present position he served for 3 years as assistant concrete control engineer and concrete control engineer for the St. Lawrence Seaway Authority of Canada. His responsibilities included inspection, material approval and acceptance, mix proportioning, and quality control. Mr. Kidd's earlier experience in concrete control included power development projects of the Hydro-Electric Power Commission of Ontario.

United States and Canadian construction practices were basically similar, although some significant differences were permitted or caused by differences in specifications. Concrete was generally mixed and placed by standard methods, and automatic type central mixing plants were used on most projects. Concrete was dumped into forms by bottom-dump buckets, and in some instances, belt conveyors were used by the Canadians to transport concrete. An air-entraining agent was used in all the concrete. The coarse aggregate was crushed rock, and the fine aggregate either crushed rock, natural sand, or a combination of the two.

The standard construction methods are not described herein. Rather, this paper concentrates on comparing the major differences in concrete construction practices of the two countries, particularly those caused by differences in specifications. Aggregate production difficulties and their relation to specification requirements of both countries are discussed fully.

AGGREGATE SPECIFICATIONS

Canada specified limits on deleterious material in the aggregate, and supplied the concrete aggregate at no cost to the constructing contractor. These aggregates were obtained under a supply contract awarded prior to the construction contract to expedite lock construction work. Tests on aggregate from potential sources were made by the Department of Public Works Laboratory in Ottawa, which performed acceptance tests on all material for the Canadian seaway work. Besides the standard freeze-thaw tests carried out for the acceptance of the aggregates along with the normal acceptance tests, freeze-thaw tests on job-mixed specimens were made regularly throughout the construc-

tion of each of the Canadian locks. Along with each freeze-thaw specimen, companion 6 x 12-in. and 8 x 16-in. cylinders were tested; this gave assurance of the durability as well as the ultimate strength of all concrete.

United States specifications (by the Corps of Engineers) placed no limit on deleterious material, but required that the aggregate come from an approved source. Both U. S. contractors purchased their aggregate from the same producer. Prior to construction, aggregates in the potential deposits were tested to determine their acceptability. Information relative to uniformity of material in the deposits, service records of material from those deposits, their location with respect to the projects, and the amount of material available was obtained. This information was considered, together with test results, in determining whether material from a given deposit was acceptable.

Gradation requirements

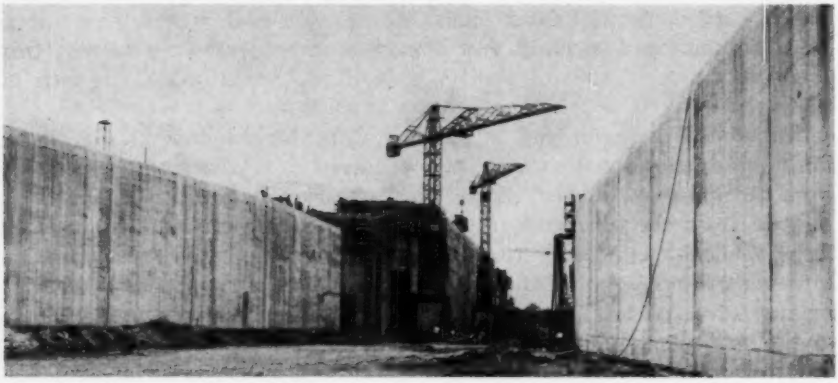
Coarse aggregate size fractions used were: 3-6 in., 1½-3 in., ¾-1½ in., and No. 4-¾ in. Gradations specified for each size fraction used by the two countries were practically identical, except for the 3–6-in. size allowed by the United States. Maximum size aggregate used by Canada was 3 in. since it was concluded that any saving realized by the use of 6-in. aggregate would be offset by the additional plant required to handle, batch, and mix this aggregate. Also fewer restrictions were put on transportation for this concrete, so that methods such as conveyor belt, chutes, etc. could be used.

The larger size coarse aggregate specified by the Corps of Engineers made possible a lower cement factor in most of the concrete for the U. S. projects (Table 1).

TABLE 1 — TYPICAL CONCRETE MIXTURES

Properties for comparison	Construction by					
	United States			Canada		
Mix No.	C8,* exterior	C9,* interior	B4.1*	St. Lambert Division†	Iroquois Division*	Beauharnois Division‡
Maximum size aggregate, in.	6	6	3	3	3	3
Water-cement ratio by weight	0.49	0.64	0.49	0.52	0.57	0.50
Cement factor, sacks (94 lb) per cu yd	3.80	2.75	4.20	4.74	4.19	5.59
Sand-aggregate ratio, percent of total aggregate	23	23	28	30	33	24
Equivalent air content, 1½ in. maximum	6.1	6.2	5.7	5.0	4.7	4.0
Slump in.	1½-2½	1½-2½	1½-2½	1½-2	1-2	1½-2½

*Coarse aggregate for these mixtures is Beekmantown dolomite.
†Coarse aggregate for mixture St. Lambert Division is Trenton limestone.
‡Coarse aggregate for Beauharnois Division is Potsdam sandstone.



Hans van der Aa

Whirly cranes in action placing concrete in Cote Ste Catherine Lock chamber. Completed approach walls in the foreground. This lock was built by Canadians.

Fine aggregate gradations used by the two agencies are shown in Table 2. The Corps of Engineers has traditionally favored finer sand. Their fineness modulus limits accordingly required the contractor to adhere to the finer limits of the specified gradation. Because of this and the more restrictive gradation limits, the sand specified for U. S. construction was slightly more difficult to produce. The U. S. aggregate producer elected to manufacture the fine aggregate from the dolomitic rock used as the source of the coarse aggregate. Considerable difficulty was experienced meeting the specified gradation because of the tendency of this rock to break down from the No. 4 size to the —100 size. Experience on this project emphasized the need for processing samples of the rock through a pilot plant prior to start of construction, to determine crushing characteristics of the rock.

Maintaining gradation limits at the mixer

On the U. S. seaway projects specifications required that the material be within gradation limits on entering the mixer. To comply, the contractor had to rescreen the coarse aggregate as it entered the batch plant storage hoppers. This was done at the top of the batch plant by two double-decked sloping, vibrating screens which operated independently. Also, all coarse aggregate except the ¾-in. size was required to pass over a rinsing and dewatering screen located at the lower end of the conveyor used to transport the aggregates to the batch plant. To make efficient use of the two double-decked rescreens, it was found necessary to load the reclaim belt with two sizes simultaneously; i.e., 6 in. and 1½ in. or 3 in. and ¾ in. When it was necessary at infrequent intervals to transport only one size of coarse aggregate to the batch plant by the reclaim belt, the amount was reduced to avoid

overloading the screens. The contractor also frequently switched from one size fraction to another when loading the storage bins to eliminate large slugs of oversize or undersize materials which were predominantly one size. The action of final screening on the U. S. projects was satisfactory to both engineers and contractor. Maintaining the specified gradation going into the mixer was not a problem and stockpiling operations were much simpler and less costly.

The Canadians achieved compliance with the desired gradation going into the mixer by three requirements, namely:

1. When the percent passing the finest screen of a specific fraction exceeded double that permitted by specifications, adjustments were made in the screening process to correct this and the ratios of the size fractions as weighed going into the mix were also adjusted to maintain the desired composite grading.

2. The fine aggregate was handled so that the fineness modulus did not vary more than 0.20.

3. All aggregate was handled so that there was no mixing of one size group with another before discharge into the mixers.

4. To comply with the above requirements it was found necessary to install rescreening and washing plants at all the divisions except at Beauharnois where because of the nature of the sandstone aggregate the less adherent properties of the fines needed only a dry rescreening plant.

AGGREGATE PRODUCTION ON U.S. PROJECTS

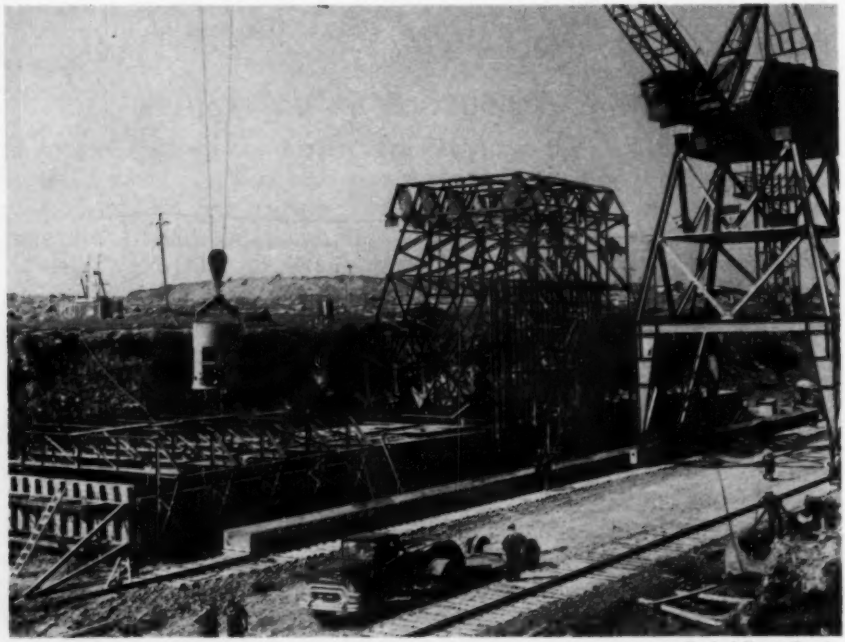
Aggregate for the U. S. projects came from a new quarry located in the Beekmantown dolomite formation near Helena, N. Y., about 12 miles away. From 5 to 12 ft of stripping was necessary and the top of rock was cleaned of all foreign material before quarrying could commence. Two Ingersoll-Rand Quarrymasters were used to drill 6-in. holes on about 14-ft centers 35 ft deep in the deposit. Sufficient holes were drilled and shot each time to produce about 100,000 tons of rock. Rock was loaded on 15-cu yd dump trucks by 3-cu yd shovels and hauled to the primary crushers.

During the life of the contract about 2,000,000 tons of aggregate were produced. Two 35-ft lifts were excavated and about half of another 35-ft lift. The total depth of the quarry on the north end therefore was finally about 100 ft. Two Telsmith primary jaw crushers were used, one 42 x 48 in. with a capacity of about 700 tons per hr, and a smaller one, 36 x 40 in. with a capacity of about 400 tons per hr. From the primary crushers, rock was further crushed and screened into the four sizes of coarse aggregate required. The various pieces of equipment and screens were so arranged that all of the material could be crushed into the smaller sizes if desired. After the 3–6-in. size was produced, the minus 3-in. material was run onto a surge pile which fed the tertiary screening and crushing plant.

Excess No. 4–¾-in. material and minus No. 4 material from the tertiary screening plant ran into a silo holding approximately 500 tons which fed the sand plant. The aggregate plant, therefore, was divided into three parts, each of which could be operated independently of the other, so that in the event of a breakdown, only one-third of the plant would be demobilized. There were no clay seams or other strata of deleterious material in the deposit, so it was unnecessary to wash any of the aggregate.

Fine aggregate

The contractor's election of the dry process in the production of fine aggregate was approved. The fine aggregate plant included at the start a screening plant, four shorthead cone crushers, and a Sturdevant classifier. The fine aggregate was dumped onto a revolving cone at the top of the classifier which separated the coarse material from the excess minus 100 material by centrifugal force. The excess minus 100 material removed amounted to approximately 10 percent of the total sand production. From time to time, additional pieces of equipment were added



Capital Press Service

Placing equipment and steel traveling form used in concreting all approach walls and lock chamber, except for curving or irregular sections, at Iroquois Lock. This form was modified three times to suit profile of wall structures situated on different bases

to the sand plant in an attempt to meet the gradation specification. Also about 2 percent moisture was added to the sand going into the stockpile at the plant to prevent segregation from the wind and stockpiling operations.

Coarse aggregate

No particular difficulty in the production of specification coarse aggregate was encountered during the life of the contract. The sizes of screens on which the material was separated were changed to correct deficiencies in gradation which occurred at the start of operations. Also, close watch was required to detect signs of screen wear which affected the gradation. The silica content of the aggregate, approximately 9 percent, caused greater wear on the screens than the contractor had anticipated. A conveyor which had been constructed under all the stockpiles of coarse aggregate and sand did not have sufficient capacity to load all material shipped. During the first year's operation, the 3–6-in. and the 1½–3-in. aggregate were therefore loaded by a steam shovel. This resulted in variation in gradation within the size which could not be corrected by final screening at the batch plant. The contractor was then required to install a separate conveyor under the stockpiles of the 3–6-in. aggregate and load the 1½–3-in. size by the conveyor used for the smaller sizes. Loading the coarse aggregate from conveyors under the stockpile eliminated objectionable variation in the gradation within the size group since the material was always removed from under the center of the pile.

Other production problems

Aggregate was produced and shipped to the project the day before initial concrete placement was started at the Dwight D. Eisenhower Lock. Because of the tight schedule and delay in producing aggregate, it was not possible to make adjustments which would normally be made prior to use of the aggregate in the concrete. To avoid delay adjustments to the plant were made while aggregate was being produced and shipped. The gradation of the fine aggregate produced at the start was nearly within the specifications. It then appeared that only simple adjustments to the plant would be necessary to bring the aggregate completely within the specifications.

Unfortunately, aggregate production was started during a dry period. Shortly thereafter, rains made the fine aggregate so wet that it was impossible to eliminate all the excess minus 100 material by use of the Sturdevant classifier. A dryer having a capacity of 150 tons per hr was installed to dry the material fed into the sand plant. This dryer successfully eliminated the difficulty from the damp material and was used intermittently throughout the production of aggregate. Although the excess minus 100 material was eliminated, the fineness modulus was still

too high and the percent passing the No. 16 sieve too low. Various combinations of screens in the screening plant were used and adjustments made to the setting of the four shorthead cone crushers in an attempt to correct the gradation and reduce the fineness modulus.

The changes in the screens and the adjustments to the cone crushers were not successful in bringing the sand consistently completely within the specifications and it was determined that another type crusher would have to be used. A roll crusher was installed with no improvement. A small hammermill was next installed with fairly successful results. As a result of the success with the small hammermill, and to increase production, the contractor installed two large hammermills. Most of the deficiencies in the gradation of the fine aggregate were corrected by their use; however, it was still not possible to consistently meet the specified gradation without curtailing production below job requirements.

The contractor, therefore, decided to use a natural blend sand which was locally available and would supply the sizes which were deficient. The use of a blend was permitted by the specifications and permission was granted for its use after tests indicated the material was acceptable. By its use, the contractor was able to meet the specified gradation and still maintain production sufficient to meet the needs of the projects. The proportion of blend used varied from about 10 percent at the beginning to a maximum of 25 percent at the finish. The blend was loaded by a Syntron feeder onto the conveyor belt leading to the sand hopper from which trucks were loaded. Use of the feeder enabled the contractor to maintain the percentage added within narrow limits. By the time the sand had been shipped to the project, conveyed to the stockpile and from there to the batch plant storage bin, it was thoroughly mixed and no segregation of blend or manufactured fine aggregate existed.

AGGREGATE PRODUCTION ON CANADIAN PROJECTS

Iroquois Lock

The aggregate for the Iroquois Lock was produced from Beekmantown dolomite, quarried 2½ miles northwest of the project. Previous to accepting this material, it was sampled and crushed in the pilot plant of the dominion government's Department of Mines and Technical Surveys laboratory. The crushing equipment required, and the gradings to be expected were indicated by this work.

Two primary jaw crushers, 25 x 40 in., supplied material to a parallel crushing and screening system. The one system supplied the aggregate exclusively for the lock concrete. Secondary crushing of this aggregate

was supplied by two Symons 4-ft standard cones and two triple-decked 4 x 12-ft horizontal Cedar Rapids screens which classified the required fractions passing the 3-in., 1½-in., and ¾-in. screens. A ¾-in. maximum material containing only the fines from the secondary crushing but none of the primary crushing supplied the sand plant. Varying amounts of this material were fed from a 600-ton silo through a Syntron feeder to a 4 x 10-ft double-decked screen located over a Sturdevant air classifier. The Syntron feeder was adjusted to provide proper screening and crushing for varying amounts of moisture in the feed material. Both top and bottom decks of the double-decked screen were divided into two different mesh sizes. This allowed a maximum of material to be screened to the required grading. The material retained on these screens fed a 4-ft shorthead cone and a 22 x 40-in. twin roll crusher which operated in parallel and in closed circuit with the feed from the silo. The Sturdevant classifier drew off the No. 200 and some No. 100 and No. 50, leaving the classified product to be stockpiled.

Beauharnois Lock

At Beauharnois, rubble from lock excavation supplied the concrete aggregates. Preliminary samples indicated preference for jaw and cone crushers. A single 48 x 60-in. primary jaw crusher broke down the Potsdam sandstone to a maximum 5-in. supply for the secondary crushers. No rescreening of material from the primary crusher was done since the amount of deleterious material was so minute that all fines would be acceptable in the fine aggregate. Two standard cones, one 5½-ft and one 4½-ft, supplied the secondary crushing for the 3-in. and 1½-in. material. Two 4-ft shorthead cones produced the ¾-in. material and sand. Each crusher was in a closed circuit with a sizing screen: twin vibrating, double-decked, 5 x 12-ft for the 3-in. material; and twin vibrating, single deck, 5 x 14-ft for the other fractions. Thus, each fraction was supplied by the fines of the previous products, but any crusher-screening combination could be cut out or put on the line to build up a stockpile of any one fraction. Since this rock was 96 percent silica, three separate screen houses isolated the dust hazard.

As previously stated, dry rescreening of all coarse aggregate was done immediately before the mixing plant on two double-decked horizontal 4 x 16-ft screens. This plant produced good coarse aggregate gradings with good particle shape, but due to the high attrition of the sandstone, the specified grading for the sand fraction was hard to meet.

There was an excess of material retained on the No. 8 sieve and next to nothing on the No. 16 and No. 30 sieves. This was aggravated by an ample amount retained on the No. 100 and No. 200 sieves and additional crushing of the No. 8 produced an excess on these sieves. The possibility of correcting this gap grading in the middle sizes with

a natural blending sand was thoroughly investigated and it was found that such a blend sand could not be located or produced economically.

A typical grading of the sand fraction produced from the Potsdam sandstone was:

Sieve size	Percent passing
No. 4	80.0
8	64.0
16	56.4
30	49.6
50	27.7
100	7.6
200	3.6
Fineness modulus	3.15

St. Lambert and Cote Ste Catherine Locks

The National Quarries also Miron and Frères Ltd., of Côte St. Michel on the island of Montreal supplied the coarse aggregate from the same source of Trenton limestone for the St. Lambert and Cote Ste Catherine locks, respectively.

The National Quarries is one of the largest commercial plants in the world with an output of 25,000 to 30,000 tons per day. The quarry has an 80-ft face which is operated on a single bench. The primary crushing was accomplished by a 54-in. Traylor gyratory crusher. The secondary crushing was done by three Symons standard cone crushers, 7½-ft, 5½-ft and 4½-ft plus a 4-ft shorthead cone, all housed in a single crushing house.

The feed for these secondary crushers was supplied from scalping screens and their product combined on a 42-in. conveyor belt and delivered to three screening towers for sizing the various fractions. This system of screening provided large stockpile areas because each pile could be extended indefinitely without interfering with each other. From these stockpiles, the lock concrete aggregates were selected, blended, and rescreened to remove excess fines. Because of the proximity of the project, the supply was made on a daily basis by trucks.

A similar operation was maintained by Miron and Frères Ltd., using for the primary a 36 x 48-in. jaw crusher. Secondary crushing was done by three standard cone crushers and four babbitless gyratory crushers. Deliveries were also made on a daily basis by trucks.

Fine aggregate for both of these locks was supplied by the Standard Lime Co. from a large natural deposit at Joliette, Quebec. A square mile of open pit was crossed by a horizontal belt conveying system. Scrapers picked up the sand from fine and coarse areas, blending them over the central belt in approximately 1:1 proportions. A central scalping screen eliminated the oversize, and the finished product was loaded into railway cars. Mechanical analysis of samples from each car provided close control in adjusting the coarse and fine blending, main-

taining the grading well within specification limits. No further washing or screening of this material was required.

CEMENT

Portland cement used on the United States projects conformed to federal specification SS-C-192, Type II. Alkali reactivity was no problem so it was not necessary to limit the alkali content of the cement. The contractors were permitted to use natural cement. On one project, the Dwight D. Eisenhower Lock, natural cement conforming to ASTM D 10-54 was used as a replacement for 25 percent by weight of portland cement. The Canadians used portland cement conforming to CSA Specification A5-1951 which is similar to federal specification SS-C-192, Type I. Both agencies required the cement to be tested prior to shipment to the project and both agencies paid the contractor for the cement separately.

MIXING AND PLACING CONCRETE

Proportioning

Canadian concrete was proportioned to produce a specified strength, which varied depending on the construction feature in which the concrete was placed. The water content of the mixture was determined from results of tests on the materials to be used in the structure. Field concrete control laboratories conducted these tests, and along with the previously mentioned comparative tests an inspection staff maintained supervision of all concreting.

For U. S. construction, a fixed water-cement ratio was used for the same class concrete having the same degree of exposure. The cement factor was varied as required to produce placeable concrete. The percentages of each size group of coarse aggregate were chosen to produce a combined grading agreeing as closely as practicable to an ideal gradation computed from a formula which is an adaptation of Talbot's formula

$$P = \sqrt{\frac{d}{D}} = \left(\frac{d}{D}\right)^{0.5}$$

Where P is the cumulative percent passing a square mesh sieve of size d for aggregate having a maximum size D. The adaptation consisted of using an exponent of 0.38 instead of 0.5. In general an exponent of 0.3 would be used with angular flat and elongated crushed aggregate and 0.5 with spherically shaped natural gravels.

The lock walls on the United States seaway project were designed as mass concrete structures permitting interior concrete to be used

in the interior of the monoliths. Interior concrete was used partly to save cement but principally to reduce the heat of hydration in the fairly massive sections. About 39 percent of all the concrete placed was interior concrete.

Both agencies required air entrainment in the concrete in about the same amount. Typical mixtures used by Canada and the United States are given in Table 1. On the U. S. projects, concrete containing 6-in. maximum aggregate (Mixtures C8 and C9) was used in more than 80 percent of the total concrete placed; C9 was used in the interior.

Mixing plants

The concrete mixing plants used by the two countries were essentially the same. Automatic plants were required by the U. S. but not by the Canadians. Canadian contractors, however, generally elected to use the automatic plant.

Transportation and placing

Canada permitted the use of belt conveyors and chutes whereas the U. S. did not because of drying out of the concrete during hot weather and segregation at the end of the chute or conveyor. Baffles at the end of the chute or conveyor will correct segregation; however, they are frequently improperly located, knocked out, or impractical to use. Chutes were permitted by the Canadians provided the concrete slid in them without excessive segregation (i.e. the slope and length of the chute were restricted). Segregation taking place on leaving the chute was corrected by various methods — baffles, hoppers, drop-boots, etc.

The U. S. required that, as far as practicable, placing of concrete should be done by bottom-dump buckets. Canada specified that the use of the bottom-dump bucket was desirable provided the bucket did not touch the forms, reinforcing, or form ties.

TABLE 2—
GRADATION FINE AGGREGATE

Sieve, U. S. Standard, square mesh	Percent passing, by weight	
	Canadian construction	United States construction
⅜ in.	100	
No. 4	95-100	95-100
No. 8	80-100	80-90
No. 16	50-85	55-75
No. 30	25-60	30-60
No. 50	10-30	12-30
No. 100	2-10	3-10
No. 200	Maximum, 5 percent	—
Fineness modulus	2.50-3.00	2.40-2.90

Canada required that concrete must be covered by subsequent layers within 60 min. The U. S. agency did not require a time limitation, but specified that placement should be carried on at such a rate that the formation of cold joints would be prevented. To prevent cold joints, placement was usually accomplished by the step method.

The thickness of layers specified by the Canadians was 12 in. and by the U. S. 20 in.

One major difference existed in the height of lift placed by the two agencies. The U. S. restricted the height of lift to 5 ft in monoliths more than 16 ft wide. The Canadian Seaway Authority did not restrict the height of lift, thus concrete was placed monolithically to heights of 37 ft and in a few forms to 50 ft.

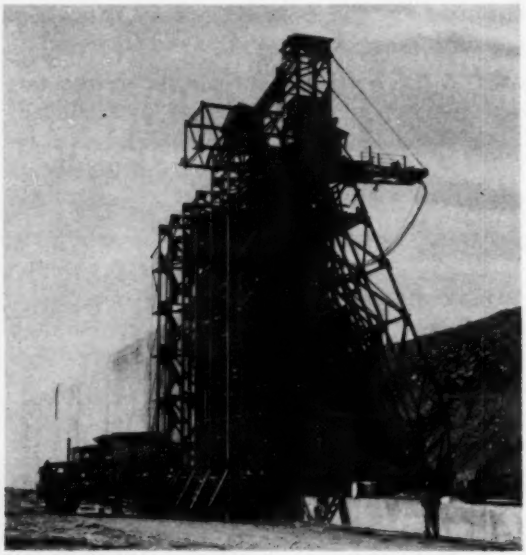
The U. S. agency required that 120 hr must elapse between lifts. No time limitation was imposed by the Canadians. This restriction and the one preceding materially affected the contractors operations. Unfortunately no comparison has been made of the projects constructed by the two agencies to determine whether the different requirements had an effect on shrinkage cracks.

CURING

Curing required by the two agencies constituted one of the major differences. The Corps of Engineers required that concrete be cured by keeping it moist for 14 days. Curing compounds were permitted only in isolated cases. The Canadians required moist curing for 7 days but curing compounds approved by the engineer could be used

Hans van der Aa

St. Lambert Lock, one of those built by Canada, shown during construction. Concrete is being placed by conveyor belt, truck, bucket, and crawler crane. All conventional wood forming was used on this project



Structural steel traveling form used by Canadians in concreting entrance walls of Cote Ste Catherine Lock facilitated placing concrete in lifts as high as 38 ft

Hans van der Aa

in lieu of moist curing. This shorter period and method of curing is more consistent with Canadian winter concreting practice and also high lift construction.

CONSTRUCTION JOINTS, SURFACE CLEANUP

The Canadians did not limit the height of the lift and construction joints on their projects, therefore, were considerably less frequent than on U. S. projects where the height of lift was limited to 5 ft. The U. S. agency required rustication strips at the construction joint on exposed surfaces. These strips successfully concealed the construction joints and broke up the wall surface to result in a more pleasing appearance. The Canadians did not require rustication strips in lock structures and approach walls where it was anticipated that the ships would mar the face during passage.

The Canadians required the surfaces on which concrete was to be placed to be cleaned of all foreign material and laitance by an approved method. The general method used was to cut the laitance by stiff brooming of the concrete surface when the laitance could readily be removed without remixing or "muddying" the whole mortar film. The dried laitance was later broomed or blown out. In the event that this practice was not carried out properly, the contractor chipped and/or bush hammered the surface. The U. S. required cleanup of construction joints either by an air and water jet or wet sand blast.

COLD WEATHER

In cold weather the Canadians required that aggregates and mixing water be treated in the manner and to the temperature required by the engineer. Concrete forms and the area in which concrete was to be placed were heated to 38F prior to, during, and after placement operations. After 7 days, protection was removed in such a manner that the exposed concrete surface would not experience a greater temperature gradient than 20F in 24 hr.

In accordance with established Corps of Engineer policy the concrete on U. S. projects was maintained at a temperature above 40F for at least 5 days and above freezing for the remainder of the 14-day curing period. The corps has considered this policy to be justified in spite of the added cost. Removal of protection was accomplished so that the surface of the concrete was not subjected to a temperature differential of more than 25F. Concrete when placed was required to have a temperature of not less than 40F, nor more than 60F. When regular placement of successive lifts was interrupted by more than 30 days from September through April, the top of lifts and bulkhead surfaces were protected by insulating with material equal to 3½ in. of lumber. This was accomplished on one project by insulating with 1 ft of straw held in place by tarpaulins. On another project the bulkheads were insulated by 1¼ in. of Styrofoam. This foamed polystyrene was obtained in strips 12 in. wide which were fastened tightly against the concrete by wood strips. This method required less maintenance than protection by straw and tarpaulins.

CONCLUSIONS

The differences in construction practice and materials enumerated had an appreciable effect on the contractors' operations. The limited delay between placing successive lifts and the limited height of each lift forced the U. S. contractors to place concrete in a number of monoliths concurrently to maintain the required schedule. The U. S. method permitted more standardization of forms thus enabling the contractor to obtain more uses per form. The Canadians could complete a monolith in shorter time and their operations therefore were more concentrated.

The 5-ft lift, coupled with the 14-day curing required by the Corps of Engineers, increased the cost of winter protection on U. S. projects. Practically all the U. S. concrete was therefore placed when heat would not be required. Production was geared to complete concrete construction during the warm weather. The Canadians, however, placed concrete through the winter months.

The physical differences between the locks constructed by the two countries provide a basis for comparison of operating efficiency, but of greater value will be the comparison of concrete durability resulting from the major differences in concreting techniques, essentially in curing methods and high lift versus low lift construction. There are undoubted advantages and disadvantages for both methods, but the final conclusions will not be made until some 50 years hence.

ACKNOWLEDGMENTS

The authors are grateful to the following for their assistance in the preparation of this paper: J. C. Kingston, General Electric Co. of Canada, formerly concrete engineer with the St. Lawrence Seaway Authority of Canada; and J. N. Mustard, assistant engineer-in-charge, Concrete Control Department, Research Division, Hydro-Electric Power Commission of Ontario, Toronto.

Received by the Institute June 30, 1958; based on papers presented by the authors at the 54th annual convention of the American Concrete Institute, Chicago, Ill., Feb. 25, 1958. Title No. 56-24 is a part of copyrighted Journal of the American Concrete Institute, V. 31, No. 5, Nov. 1959 (Proceedings V. 56). Separate prints are available at 60 cents each.

American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by February 1, 1960, for publication in the June 1960 JOURNAL.

Title No. 56-25

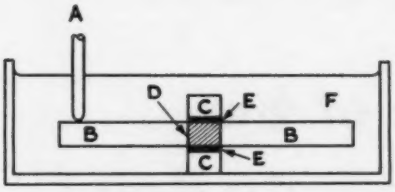
Strength of the Cement-Aggregate Bond

By K. M. ALEXANDER

Data from approximately 1000 determinations of portland cement-aggregate bond strength and portland cement paste strength are presented and analyzed. It is shown that at early ages, large differences can exist between the strength of the bond formed between the same cement and different aggregates. No evidence was found of significant cement-aggregate bond strength differences between different samples of the same rock type. Cement-aggregate bond strength decreases with increasing water-cement ratio and alkali content, and increases with increasing age. All cement-aggregate bond strengths observed here for sawn aggregate surfaces were considerably lower than the strength of the adjacent portland cement matrix at 7 and 28 days. On the basis of these data explanations are suggested for some aspects of the process of tensile failure of plain concrete.

PLAIN CONCRETE CAN BE REGARDED AS A CHAIN comprising three links — cement paste, aggregate, and cement-aggregate bond. A knowledge of the strength of each link, and the factors which control the strength of each link, will enable the weakest point to be located, and attention to be concentrated on eliminating the cause of the weakness. The three links have not, however, received equal attention from research workers. In particular, the large volume of work relating to the properties of the cement matrix can be contrasted with the limited number of investigations designed to show how strongly the matrix adheres to the aggregate.

In previous studies of the cement-aggregate bond in France, Farran[1,2,3] focussed attention on the mineralogical aspects of bond. Farran[1] includes the results of a few tests on the strength of the bond between mortars of normal consistency, and polished aggregate surfaces. An earlier investigation of cement-aggregate bond was carried out at the Ecole Nationale des Ponts et Chaussees, and reported by Mesnager.[4]



A PLUNGER FOR APPLYING LOAD.
B HARDENED PASTE.
C CLAMP.
D AGGREGATE.
E PLASTIC LINING.
F WATER.

Fig. 1—Apparatus for measuring cement-aggregate bond strength

Measurements were made of the bond between cement paste and broken, sawn, and chipped surfaces of a number of different rock types. Several tests to determine the adhesion between concrete blocks and the foundation rock of a dam are reported by Bourriot,[5] while Kessler and Anderson[6] have measured the strength of the bond between stone-setting mortars and a range of building stones.

Experiments described below comprise the preliminary stage of an investigation into factors controlling cement-aggregate bond strength. The bond to sawn surfaces of different rock types is compared with the cohesive strength of the adjoining matrix. The variation of cement-aggregate bond strength between points located in a single pebble is then compared with the difference between bond to pebbles from different sites in the same formation or flow, and with the difference between the bond to different samples of the same rock type selected from flows in different areas. Finally, the dependence of bond strength on the properties of the matrix is studied, the relationships between bond and water-cement ratio, and the variation of bond with age being determined. On the basis of these data a possible explanation is suggested for some aspects of the process of tensile rupture of plain concrete.

TESTS

Cement-aggregate bond strengths were determined by measuring the transverse load required to rupture the bond between ½-in. sawn aggregate cubes, clamped as shown in Fig. 1, and 3-in. projecting beams of paste. Surfaces which were vertical during molding were located horizontally when testing. The residual paste ends were subsequently placed in the clamp and the moduli of rupture were determined by an analogous procedure. Aggregate cubes were bonded in the saturated surface-dry condition. The cement-aggregate composite specimens were cured in the molds for 24 hr and subsequently under water until tested, a temperature of 73 F being maintained throughout.

All tests were performed with the specimens and clamp under water since earlier tests by Alexander[7] showed that rapid loss of flexural

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strength on exposure to air would be a major source of error. Trial experiments also indicated that approximately 40 determinations of bond strength would be required if an aggregate was studied under only one set of conditions, and that not less than 20 replications per point should be made where the aggregate was used over a range of different conditions.

A Type I portland cement with a total alkali content (expressed as Na_2O) of 0.41 percent was used. Owing to the relatively high strength of the cement-aggregate bond, aggregates of medium or high strength (Table 1, Column 4) were selected.

RESULTS AND DISCUSSION

7-day bond strengths — General remarks

The strengths of the bonds formed between portland cement paste and the sawn surfaces of ten different aggregates were determined at 7 days, and are given in Table 1. Values quoted are the means of 40 determinations. A paste with the water-cement ratio of 0.28 by weight was found to be suitable for this stage of the work. At 7 days this paste had a modulus of rupture of approximately 2000 psi.

It will be seen from Table 1 that mean bond strengths ranging from 545 to 1503 psi were observed, and that the coefficients of variation were high in all cases. The form of the frequency distribution of bond

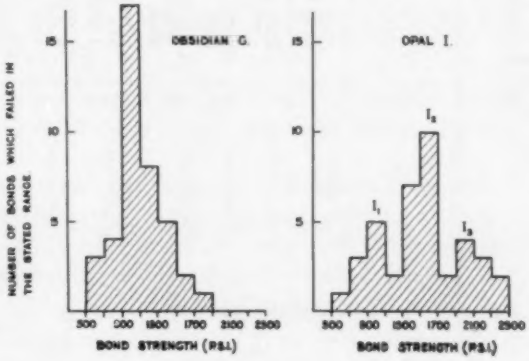

Fig. 2—Histograms of 7-day cement-aggregate bond strength illustrating the difference between a typical homogeneous aggregate (right) and a nonuniform material (left)

TABLE I—DESCRIPTION OF AGGREGATES AND GENERAL INFORMATION ON 7-DAY CEMENT-AGGREGATE BOND STRENGTHS

No.	Rock type	Source	Modulus of rupture of aggregate, $\overline{X}_{20}$ psi	7-day bond strength*		Multiple range test †	Mean bond strength difference expressed as a percentage of mean bond strength	
				Modulus of rupture, $\overline{X}_{40}$ psi	Coefficient of variation, percent		a Between opposite faces of same cube	b Between faces of different cubes
A	Rhyolite tuff	Castle Rock, Colo.	—	545	26		—	—
B	Pitchstone	Owyhee Dam, Ore.	2530	913	20		19.7	21.9
C	Graywacke	Lone Tree Creek Marin County Calif.	2480	1002	21		—	—
D	Glassy andesite	Loveall Canyon Rd., Loveall Canyon, Calif.	5800	1047	23		19.2	26.5
E	Glassy obsidian	Glass Mountain Lake County, Ore.	6990	1079	25		25.0	27.7
F	Glassy obsidian	Warner Mountains, Modoc County Calif.	6510	1090	24		24.3	28.7
G	Glassy obsidian	St. Helena, Napa County, Calif.	6240	1098	22		22.6	25.1
IIa	Opaline chert	Berkeley Hills, Calif.	5450	1144	23		18.8	22.0
IIb	Opaline chert	Berkeley Hills, Calif.	5450	1146	26		22.5	28.6
I	Opal	Coulee Opal, near Quincy, Wash.	—	1503	31		25.9	41.4
							$\overline{X}_a$: 22.25	$\overline{X}_b$: 27.74

*Modulus of rupture of paste at 7 days: $\overline{X}_{179} = 1994$ psi.

† The bond strengths of any two aggregates not connected by the same vertical line in this column are significantly different at the 5 percent level of significance. Any two means connected by the same line are not significantly different.[8]

strengths is illustrated by the histograms in Fig. 2, in which the examples quoted are (a) a typical homogeneous rock and (b) a nonuniform material.

Differences between the cement-aggregate bond strengths of different rock types

Table 1 shows that extremely large differences can exist between the strengths of the cement-aggregate bonds formed by different rock types. Analysis of variance shows that the differences between the mean bond strengths in Table 1 are significant at the 1 percent level, and that the between-sample differences are still significant, at the same level, even if the extreme values for Aggregates A and I are omitted.

Differences between the cement-aggregate bond strengths of different samples of the same rock type

The variations in bond strength encountered with the same rock type will be considered in several stages. To begin with, the variations between bond strengths measured at points ½ in. apart will be compared with variations between points up to 6 in. apart in the same pebble. Additional variations introduced by selecting material from points up to 20 in. apart in the same boulder, from different parts of the same formation (or flow), and from different formations (or flows) of the same rock type will then be considered.

Variations of bond strength between points up to 6 in. apart in the same pebble

The variations in bond strength between points separated by up to 6 in. will be assessed by comparison with variations between points known to have been ½ in. apart in the original pebble. Confining attention to bond strength differences observed with the same batch of cement paste, the "½ in. apart" and "0 to 6 in. apart" variations can be compared by calculating the following quantities.

(a) 1/2 in. apart variations—Calculated from the mean bond strength differences between opposite faces of each of the 20 cubes cut from each pebble.

(b) 0 to 6 in. apart variations—Calculated from the mean bond strength differences between faces located on different cubes cut from the same pebble.

The observed values of (a) and (b), expressed as percentages of the corresponding mean bond strengths, are given in Table 1. Values for Aggregates A and C have been omitted since the cubes for these materials were not cut from a single pebble of each rock.

It will be seen from Table 1 that the differences between the bond forming ability of small areas which were up to 6 in. apart in the

pebble are not much greater than the differences between similar sized areas only ½ in. apart in the same pebble. The average increase in variation associated with the change from ½ in. apart, to random separations of 0 to 6 in. is equivalent to 25 percent of the "½ in. apart" variation. However, although this increase is small, it is present in all the 7-day values for single pebbles (Table 1) and, in addition, the mean increase ($\overline{X}_b - \overline{X}_a$) for all materials studied here is significant at the 5 percent level.

Variations of bond strength between points up to 20 in. apart in the same boulder

The evidence available here suggests that the variations in bond strength between surfaces derived at random from within a 20-in. boulder of reasonably uniform composition, are probably no greater than the corresponding variations for the smaller 6-in. pebbles. The following observations support this statement.

(a) Bond strengths for Cherts Ha and Hb were measured on cubes from two 6-in. pebbles broken from the same 20-in. boulder. The difference between the two means is not significant.

(b) The batch of cubes from which the mean bond strength ($\overline{X}$ = 1002 psi) of the graywacke was determined, contains representatives from ten different 4- to 6-in. pebbles which originally comprised a single 20-in. boulder. Although all other mean bond strengths in the region of 1000 psi were calculated from cubes cut from single 6-in. pebbles, all but one of the variances and corresponding coefficients of variation are greater than the value for Graywacke C, while even the one smaller variance is only slightly lower.

Variations of bond strength between different points in the same rock flow

The 20 cubes used in determining the bond strength of Rhyolite A were cut from 20 separate water-worn pebbles. The coefficient of variation for the bond strengths is strictly comparable with the value obtained with all the other aggregates. In addition, the mean bond strength difference between opposite faces of the same cube, and between the faces of different cubes, are similar. When expressed as percentages of the mean bond strength, as in Table 1, values of a = 26.4 percent and b = 27.5 percent are obtained. The increased difference observed when comparing surfaces ½ in. apart with those derived from separate stream-bed pebbles, is equivalent to only 4 percent of the ½ in. (or a) value, compared with the previously calculated figure of 25 percent for random separations of 0 to 6 in. This 4 percent increase in the variation is in fact the lowest observed here. There is thus no evidence at this stage to suggest that appreciably greater differences exist between the bond forming ability of surfaces cut from widely spaced points in the formation or flow, than between points less than 6 in. apart.

Variations of bond strength between different flows of the same rock type

Application of Duncan's multiple range test[8] shows that, at the 5 percent level, any two means not connected by the same vertical line in Column 7 of Table 1 are significantly different, and any two means connected by the same line are not significantly different. It will be seen that although the three samples of obsidian were obtained from different rock flows, even the two most extreme mean bond strengths for this rock type differ by less than 2 percent, and that this difference lies well within the range that is considered nonsignificant. This further confirms the hypothesis that as far as unweathered rock with uniform physical characteristics is concerned, the cement-aggregate bond strength estimated from a sufficient number of replications on a single pebble is characteristic of the rock type, regardless of where the pebble is selected.

Reactive versus inert aggregate

Regarding the effect of reactivity of the aggregate, none of the estimates of the 7-day bond strength for potentially reactive volcanic rocks are significantly greater than the value for inert Graywacke C. The means for two of the volcanic materials (A and B) are in fact below that of the graywacke, the difference being significant for Aggregate A. On the other hand, the estimate of bond strength obtained for Opaline Material I is significantly greater than that for Graywacke C, while the estimates for the other two samples of opaline rock, Ha and Hb, are considerably greater than that for Graywacke C, though slightly below the level required for significance.

The provisional conclusion is that those properties which cause certain acid volcanic rocks to be classified as potentially reactive do not necessarily lead to the formation of a stronger bond with low alkali cement at 7 days, whereas, under the same conditions, the presence of reactive opaline material may result in increased bond strength.

THE PROCESS OF RUPTURE

At this stage it is appropriate to consider the question of the site at which rupture occurred in the 7-day specimens. Comparison of the high strengths of the aggregates with the paste and bond strengths (Table 1) suggests that failure through the aggregate is highly improbable.

So far as preferential rupture of the bond or paste is concerned, the decision is not as clear cut. The point is illustrated by Fig. 3, in which the histogram for 179 estimates of paste strength is compared with a histogram obtained by combining the 120 estimates of bond strength for the three samples of obsidian (E, F, and G). These obsidians develop

TABLE 2—DEPENDENCE OF 7-DAY BOND AND PASTE STRENGTHS ON WATER-CEMENT RATIO (AGGREGATE: GRAYWACKE C)

W/C by weight	Bond strength, $\overline{X}_{80}$, psi	Modulus of rupture of paste, $\overline{X}_{40}$, psi	Bond strength / Paste strength, percent
0.280	1002	1994	50
0.315	998	1457	69
0.350	620	1025	60
0.500*	313	511	61
0.750†	112	253	44

*2 percent bentonite present

†4 percent bentonite present

7-day bond strengths similar to those of the majority of aggregates studied here. Although rupture of the bond is again the more likely event, the overlap of the frequency distributions is sufficient to cause preferential failure of the paste in a few instances. The limitations of the data on which Fig. 3 is based are such that only a rough estimate can be made of the proportion of cases in which this occurs. This approximation indicates that less than 2 percent of the failures are attributable to rupture of the paste bonded to Obsidians E, F, and G.

Effect of bond formation and rupture on the properties of the aggregate surface

An experiment was performed to see if the bond-forming ability of an aggregate surface was greatly affected by a cycle of bond formation and rupture. A strong rock (Obsidian G) was selected which would be unlikely to sustain physical damage during the process of rupture of the bond. Twenty cubes of aggregate from which hardened paste had been separated during bond strength tests were soaked under water

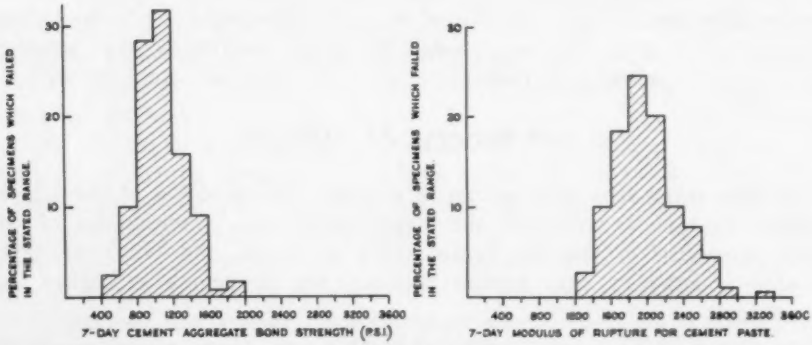
PERCENTAGE OF SPECIMENS WHICH FAILED IN THE STATED RANGE
30
20
10
400 800 1200 1600 2000 2400 2800 3200 3600
7-DAY CEMENT AGGREGATE BOND STRENGTH (PSI)
PERCENTAGE OF SPECIMENS WHICH FAILED IN THE STATED RANGE
30
20
10
400 800 1200 1600 2000 2400 2800 3200 3600
7-DAY MODULUS OF RUPTURE FOR CEMENT PASTE

Fig. 3—Histograms for the frequency distribution of cement-obsidian bond strengths and cement paste strengths at 7 days

Note: The bond and paste strength distributions show similar departures from the normal. Both are positively skewed to a significant extent, and both are peaked, although the kurtosis is significant only with bond strength.

for 24 hr, and again bonded to cement. The orientations of the aggregate cubes during the first molding had been noted, and were reproduced in the repeat bond tests.

It was found that the strengths of the repeat bonds ($\overline{X}_{40}$ = 881 psi) were, on the average, 20 percent lower than the original values ($\overline{X}_{40}$ = 1098 psi), and that the coefficient of variation remained unchanged at 22 percent. The correlation coefficient for the 40 pairs of initial and repeat bond is low (+0.23) and nonsignificant.

Effect of water-cement ratio and age on strength of the cement-aggregate bond

The inert aggregate, Graywacke C, was used for this work. Ten 4- to 6-in. pebbles were broken from a 20-in. boulder, and sawn into ½-in. cubes. One cube from each pebble was included in each batch, making a total of 20 determinations of bond strength for each set of conditions described below.

Water-cement ratio-bond strength relationship– The 7-day cement-aggregate bond strengths and the corresponding paste strengths for the water-cement ratio range of 0.28 to 0.75 by weight (Table 2) show that, as in the case of paste strength, the strength of the cement-aggregate bond decreases with increasing water content. Within the error of the experiment, there are straight line relationships between the proportion of cement present

$$\frac{\text{absolute volume of cement}}{\text{absolute volume of cement + volume of water*}}$$

and the logarithm of either bond or paste strength (Fig. 4), and, further, the lines are parallel under the conditions of experiment applying here.

*Air content negligible.

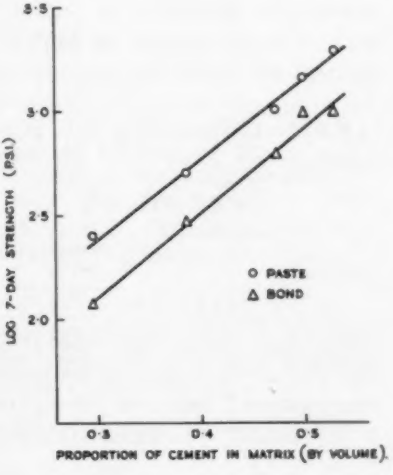

Fig. 4—Effect of W/C on strength. Relationship between proportion of cement, P, in paste and logarithm of either bond or paste strength

$$P = \frac{\text{absolute volume of cement in paste}}{\text{absolute volume of cement + volume of water in paste}}$$

Since the bleeding of pastes with water-cement ratio of 0.50 to 0.75 was minimized by the addition of 2 and 4 percent of bentonite, respectively, a preliminary check was made to ensure that this amount of additive would not seriously affect the results. For this check the bond and paste strengths at a water-cement ratio of 0.35 by weight were determined both for the straight cement, and for cement containing 4 percent of bentonite. Seven-day bond and paste strengths, with bentonite present, were 816 and 897 psi, respectively, compared with 620 and 1025 psi for the reference batches. It appears, therefore, that bentonite had little influence on the data for wetter mixes, since the effect would be to make the bond and paste strength converge, whereas a general divergence was observed here as water-cement ratio and the proportion of bentonite present were increased. The increase of 30 percent in the bond strength at a water-cement ratio of 0.35 can be attributed to the possibility that the extra work needed to compact the stiffened paste would cause the cement particles to be forced into more effective contact with the aggregate surface, thereby reducing the water-cement ratio in the interface. Similarly, the reduction of 10 percent in the paste strength could be due to incomplete compaction of the bulk of the paste. Neither of these effects should operate to any great extent at the higher water-cement ratios since the proportion of bentonite added was adjusted to give, as far as possible, the same ease of compaction at each higher water content.

It will be seen from Table 2 that as the water-cement ratio is increased from approximately 0.32, the 7-day bond strength falls from about 70 percent of the paste strength, to a value of the order of 45 percent at a water-cement ratio of 0.75 by weight. It is possible, therefore, that with a smooth surfaced aggregate, such as a water-worn gravel, the likelihood of rupture along the cement-aggregate interface will be much greater at high water-cement ratios, and a higher percentage of the paste strength would be wasted. Arguing along the same lines it would be expected that in zones of high water-cement ratio, as for example near the under surfaces of coarse aggregate pebbles in a mix subject to bleeding, there should be a relatively high proportion of pebbles for which the under surface separates cleanly from the hardened paste at rupture, even though bleeding may not have been sufficiently pronounced to cause the formation of

TABLE 3—DEPENDENCE OF BOND AND PASTE STRENGTH ON AGE (WATER-CEMENT RATIO = 0.28 BY WEIGHT)

Aggregate	Bond strength, $\bar{X}_{20}$ psi			
	1 Day	3 Day	7 Day	28 Day
Rhyolite A	—	—	545	803
Graywacke C	315	792	1002	1703
Obsidian G	—	—	1098	1495
Chert Ha	—	—	1144	1777
Paste (modulus of rupture)	767	1330	1994	2052

water-filled lenses or pockets. The importance of reducing bleeding to a minimum is thus emphasized.

Dependence of bond strength on age —Bond strengths for a series of four aggregates were measured at 7 and 28 days, additional measurements at 1 and 3 days being made for one member of the series. At least 20 determinations were made under each set of conditions. A constant water-cement ratio of 0.28 was used in this section of the work. Bond strengths, and the corresponding paste strengths are given in Table 3. The results shown in this table suggest that bond strength and paste strength develop according to different patterns. With Graywacke C for example, the bond strength is equivalent to approximately half the paste strength even at 1 day, and remains at approximately this level up to and including 7 days. However, between 7 and 28 days the bond strengths of this and the other three aggregates gain steadily on the paste strength.

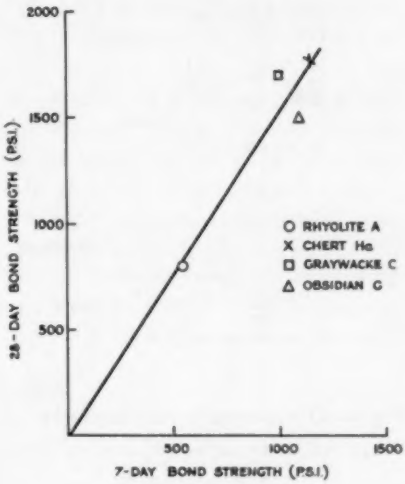

Fig. 5—Relationship between 7- and 28-day cement-aggregate bond strength

Between 7-28 days the strength of the bond formed by each aggregate increased by roughly the same proportion, regardless of whether the aggregate formed a strong or relatively weak bond with portland cement (Fig. 5). The following approximation holds under the present conditions of experiment.

$$\text{7-day bond strength} \times 1.5 = \text{28-day bond strength}$$

It is possible that the low rate of development of bond strength up to 7 days, coupled with differences between the bond strengths of various rock types, may cause apparently anomalous differences between the 7- and 28-day strengths of concrete. This could be the case for example when a concrete with an aggregate which develops adequate bond strength by 7 days is compared with one in which bond strength does not equal a significant proportion of paste strength until 28 days.

BOND TO CRUSHED AGGREGATE

At a water-cement ratio of approximately 0.3 by weight, nearly all the 7-day bond strengths measured exceeded 50 percent of the strength of the adjacent paste, while indications are that the 28-day bond

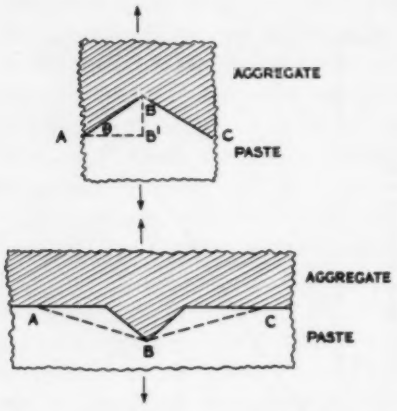

Fig. 6—Diagrammatic representation of irregularities on surface of crushed stone coarse aggregate

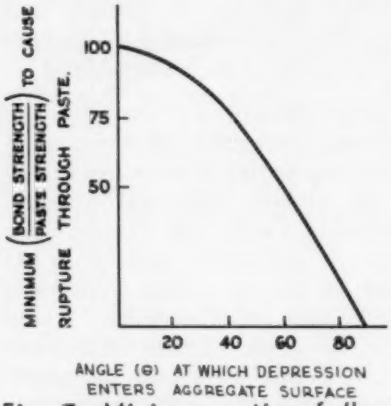

Fig. 7—Minimum ratio of (bond strength) / (paste strength) to cause preferential rupture through paste at site of depression entering aggregate surface at an angle of θ deg

strengths generally equal 70 to 85 percent of the paste strength. In addition, there is evidence to suggest that bond strength may not fall below this percentage of paste strength until water-cement ratios of 0.5 to 0.6 are exceeded. It appears reasonable, therefore, to accept that bond strength is in general equivalent to quite a high percentage of paste strength. If this is the case, then it should be noted that zones of preferential rupture through the paste can readily occur if even minor irregularities are present in the aggregate surface. There is no need to postulate that mechanical interlocking takes place.

Consider the example of a hardened paste bonded to a sawn aggregate surface in which a V-shaped groove has been cut. If the system is in tension as shown for the cross section illustrated in Fig. 6a, rupture along the whole area of interface represented by ABC would be possible if the area of paste represented by $AB'C$ has a tensile strength greater than that of the combined areas of bond represented by $AB + BC$. Conversely, in the absence of complicating factors, rupture through the Paste AC should occur when:

$$AB \cdot S_b > AB' \cdot S_p$$
$$S_b > S_p \cos \theta$$

where S_b and S_p are the bond and paste strengths, respectively. If this relationship is valid, then, expressing bond strength as a percentage of paste strength, the minimum percentage which should cause rupture through the paste will vary in the manner shown in Fig. 7. From Fig. 7 it will be seen that a value of θ in the region of 30 deg should cause

rupture through the paste if the bond strength is 85 percent of paste strength, while a bond strength equal to 50 percent of paste strength requires a θ of approximately 60 deg to cause preferential failure through the paste.

A similar line of reasoning can also be applied to irregularities which project from the aggregate surface as in Fig. 6b, where typical "short-circuit" type of preferential failure through the paste could occur along paths indicated by the broken Lines AB and BC. It appears therefore that the bond strengths observed here are in many cases sufficiently high to cause a good deal of preferential rupture through the paste which is bonded to an irregular surface such as that of a crushed pebble, even when the tensile stress is acting at right angles to the general plane of the aggregate surface.

CONCLUSIONS

Since such a vast range of combinations of cements and aggregates is possible, and since each combination could be studied under innumerable conditions of curing, temperature, age, water-cement ratio, surface roughness of aggregate, etc., it is unlikely that a single investigation could lay down rigid rules which would apply in all possible circumstances. The conclusions summarized below should therefore be applied with this limitation in mind. It is probable, for example, that Conclusion 4, relating to water-cement ratio, will have fairly general application, whereas Conclusion 2, on within-aggregate differences, is stated in terms of the restricted range of materials studied thus far, and may not apply generally.

(1) At 7 days, large differences can exist between the strength of the bond formed between the same portland cement and different strong, uniform aggregates.

(2) No evidence was found of significant differences of bond strength between different samples of the same rock type.

(3) Reactivity of aggregate is not necessarily associated with an increased rate of bond strength development.

(4) The strength of the cement-aggregate bond decreases with increasing water content, at a rate which corresponds to an approximately linear relationship between the proportion of cement in the paste, and the logarithm of the bond strength.

(5) In general, the 28-day strength of the cement-aggregate bond was found to be 50 percent greater than the 7-day value.

(6) All cement-aggregate bond strengths observed were lower than the strength of the adjacent portland cement paste. However, since the two quantities are not separated by too large a margin, the presence of even slightly angular projections or depressions on the surface

of an otherwise smooth aggregate pebble could cause the mechanism of tensile failure to change from preferential rupture of the bond to preferential rupture through the paste in the region of the surface irregularity.

(7) That bleeding of concrete could affect the proportion of cement-aggregate bond failure when plain concrete is broken in tension can be deduced from the data.

ACKNOWLEDGMENTS

This investigation was carried out at the Stanford Research Institute, Menlo Park, Calif., as part of its International Fellowship Program. The author is indebted to SRI for the award of the Fellowship, and for the wide range of facilities made available. Acknowledgment is also due to the Industrial Finance Department of the Commonwealth Bank of Australia for acting as sponsors. In addition, the author expresses his thanks to the U. S. Bureau of Reclamation and the California Academy of Sciences for supplying many of the aggregates studied here. The assistance of Prof. R. E. Davis of the University of California, G. Schlocker of the U.S. Geological Survey, and W. Beatty of SRI is also acknowledged in this same connection.

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Received by the Institute Sept. 30, 1958. Title No. 56-25 is a part of copyrighted Journal of the American Concrete Institute, V. 31, No. 5, Nov. 1959 (Proceedings V. 56). Separate prints are available at 50 cents each.

American Concrete Institute, P.O. Box 4754 Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by February 1, 1960, for publication in the June 1960 JOURNAL.

Title No. 56-26

Critique of Current Methods of Varying Prestressing Moment in Pretensioned Prismatic Beams

By JAMES R. LIBBY

Three methods currently used to vary the prestressing moment in the construction of prismatic pretensioned concrete flexural members are: bond prevention method; deflected pretensioned tendon method; and the combination of pretensioned and post-tensioned tendons. Each of these methods is described, and its limitations discussed. Comparison is made between the results which can be achieved with each method on a bridge stringer representative of the type commonly used in domestic composite bridge construction. Conclusions are made regarding the applicability of each of the methods and the research required to better apply the methods.

THE DEAD WEIGHT OF A SIMPLE prismatic prestressed concrete beam is acting at the time the member is prestressed. This is evident from the fact that the beam cambers due to the prestressing and then is only in contact with the soffit form at the beam extremities. Therefore, assuming elastic action, actual stresses in the concrete of the beam are equal to the algebraic sum of the stresses due to prestressing and due to the dead weight of the beam, at every section. The variation in the top and bottom fiber stresses of a simple prismatic beam prestressed with a straight tendon is illustrated in Fig. 1.

If, for the beam in Fig. 1, the maximum permissible concrete stresses are assumed to be 200 psi tension in the top fiber and 2000 psi compression in the bottom fiber under dead load plus prestressing, it is apparent that the beam is prestressed as highly as possible with a straight tendon. Furthermore, assuming no tensile stresses are to be permitted in the bottom fiber under the action of full design load, it will be seen that 75 percent of the stress due to prestressing in the bottom fiber at the center line is available for the superimposed load and 25 percent is required for the dead load of the beam alone. It is

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significant that the controlling fiber stresses in the elastic range occur at the end of the member where the moment due to dead and live loads is zero rather than at the center of the beam where moments due to dead and live loads are maximum.

It is axiomatic in structural engineering that the dead load and dead load moment of a flexural member itself become a progressively larger proportion of the total load and total moment which the member can resist, as the span of the member is increased. For this reason, the limitation of a member's flexural capacity due to the prestressing stresses at the end of the member is more significant for long span members. Prestressed beams of less than 50-ft span are only rarely seriously limited by the stresses at the ends while beams 75 to 100 ft may be affected greatly by this characteristic of straight pretensioned tendons.

Reconsidering the stress diagrams of Fig. 1, it should be apparent that if the prestressing force were more eccentric or larger near the center of the beam than near the ends, the reduction in beam capacity due to dead load of the beam alone could be reduced and perhaps eliminated since the actual stress in the bottom fiber at the center line could be increased to the maximum value permitted without exceeding permissible values at the ends of the beam. This can be accomplished in pretensioned prismatic beams by several methods:

1. Preventing a specific number of tendons from bonding to the concrete at the immediate ends of the member. This reduces the eccentricity and magnitude of prestressing force at the ends of the beam (*bond prevention method*).

2. Arranging all or part of the pretensioning tendons in such a manner that they are more eccentric in the center portions of the beam than at the ends (*deflected or draped tendons*).

3. Providing a few curved post-tensioned tendons to supplement straight pretensioned tendons (*combined pretensioned and post-tensioned tendons*).

Advantages and disadvantages of each of these methods are illustrated subsequently by comparing the results which can be achieved with each method on the American Association of State Highway Officials-Prestressed Concrete Institute (AASHO-PCI) standard bridge beam, Type III. Limitations of each method are also discussed.

Use of nonprismatic members is another means of overcoming the restriction of beam capacity by the stresses at the ends of members prestressed with straight tendons. The use of completely post-tensioned members is another obvious method which may be used. Use of these methods is beyond the scope of this discussion.

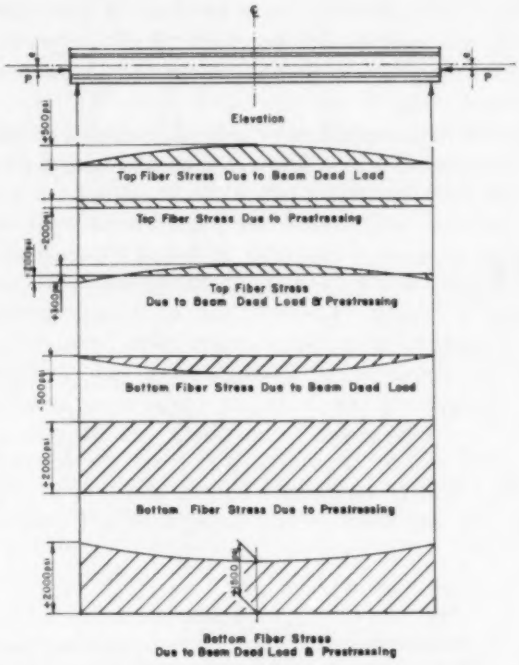

Elevation
Top Fiber Stress Due to Beam Dead Load
Top Fiber Stress Due to Prestressing
Top Fiber Stress
Due to Beam Dead Load & Prestressing
Bottom Fiber Stress Due to Beam Dead Load
Bottom Fiber Stress Due to Prestressing
Bottom Fiber Stress
Due to Beam Dead Load & Prestressing

Fig. 1. — Stress distribution in top and bottom fibers of a simple prismatic beam prestressed with straight tendons

Note also that the maximum economy which can be obtained with a fully post-tensioned flexural member may require a beam cross section significantly different from that required for maximum economy using pretensioned or combined pretensioned and post-tensioned construction. This is primarily true in long span beams because larger post-tensioning forces can be concentrated in smaller bottom flanges than is possible with pretensioned tendons, since sufficient area must be provided around each pretensioning tendon to allow the tendon to develop transfer bond near the ends. Discussion of this factor, as well as depth of construction and other fundamental forces affecting the economy and efficiency of the beam cross section are also beyond the scope of this paper.

METHOD I — VARIATION OF THE PRESTRESSED MOMENT BY PREVENTION OF BOND

In French technical literature a pretensioned member is said to be prestressed "by bond," in contrast to a post-tensioned member in which the tendons have end anchorages and are prestressed by bearing stresses. "By bond" describes the method of construction well and reminds one that if the pretensioned tendons were not bonded to the concrete they obviously could not prestress the concrete. It is also apparent that if

some effective and safe method of preventing bond can be employed, the magnitude and eccentricity of the prestressing force can be varied by applying the bond prevention technique according to a particular layout.

The effectiveness of this procedure can be illustrated by considering the AASHO-PCI Type III bridge beam prestressed as in Fig. 2. The top and bottom fiber stresses which would result from a stress of 13,000 lb per tendon for the 50 tendons positioned as indicated in Section A-A, as well as the stresses which would result if bond were prevented on the 14 tendons indicated in Section B-B, are summarized in Table 1. Table I shows that by simply preventing bond on 14 tendons, the top fiber tensile stresses have been reduced 268 psi and the bottom fiber compressive stresses have been reduced 810 psi.

Assuming this stringer were being used on a 70-ft span, the stress in the top and bottom fibers resulting from dead load of the girder alone at 3.5, 7.0, and 35.0 ft from the support would be as summarized in Table 1. Because a length of 1-3 ft is required for a tendon to fully bond, and because additional length is required for a concentrated pre-

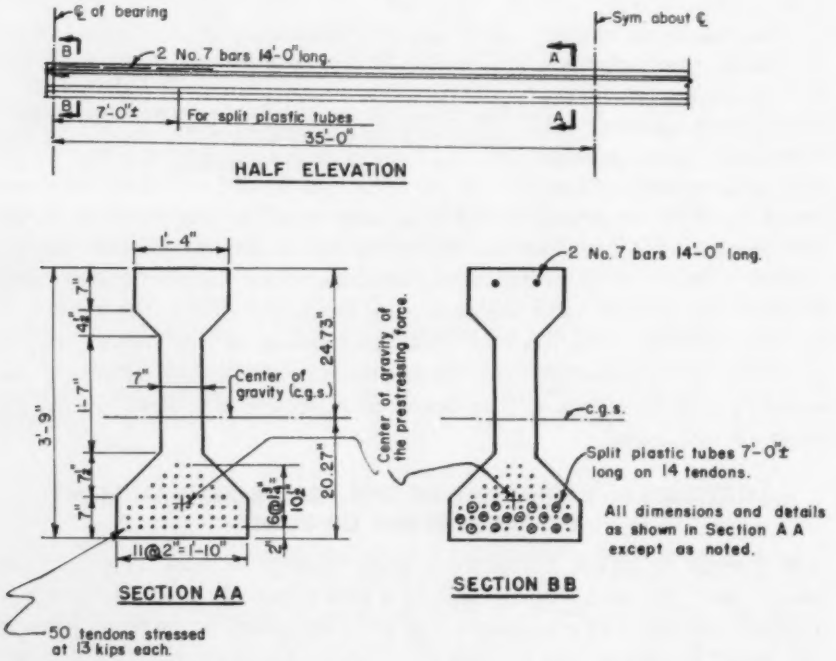

℄ of bearing
Sym. about ℄
2 No. 7 bars 14'-0" long.
7'-0"±
For split plastic tubes
35'-0"
HALF ELEVATION
1'-4"
2 No. 7 bars 14'-0" long.
Center of gravity (c.g.s.)
Center of gravity of the prestressing force.
c.g.s.
Split plastic tubes 7'-0"± long on 14 tendons.
All dimensions and details as shown in Section A A except as noted.
11@2"=1'-10"
SECTION AA
SECTION BB
50 tendons stressed at 13 kips each.

Fig. 2 — AASHO-PCI bridge stringer, Type III, pretensioned by bond prevention method

stressing force to completely distribute, the tensile stresses in the top fiber due to prestressing do not fully develop immediately at the end of a member but occur at some distance from the end of the beam. Top and bottom fiber stresses resulting from the combined action of prestress and dead load are summarized in Table 1 for different distances from the center line of bearing for 50 tendons as well as for 36 tendons. Assuming bond is prevented on the 14 tendons for 7 ft from each end and that the full effects of the prestressing due to the 36 tendons and 50 tendons develop at distances of 3.5 and 7.0 ft from the center line of bearing respectively, two #7 nonprestressed bars would be required to carry the tensile force in the top fibers of the beam since the unit tensile stresses are greater than could be permitted without reinforcing. The amount of nonprestressed reinforcing is controlled by the stresses 7 ft from the center line of bearing.

Hence, it can be seen that by preventing bond on 14 tendons for a distance of about 7 ft at each end and, in addition, providing two #7 bars about 14 ft long at each end, the concrete stresses at the end of the member can be kept within tolerable limits without resorting to deflected tendons or post-tensioning.

It should be mentioned that placing additional tendons higher in the section will increase the bottom fiber stresses due to prestressing only slightly and will slightly reduce the initial tensile stresses in the

TABLE 1—STRESSES IN AASHO-PCI TYPE III BRIDGE BEAM STRESSED WITH STRAIGHT TENDONS AND BOND PREVENTION

Type and location of stress	50 tendons, as shown in Section A-A, Fig. 2		50 tendons, as shown in Section B-B, Fig. 2 (bond prevented in 14)	
	Top fiber stress, psi	Bottom fiber stress, psi	Top fiber stress, psi	Bottom fiber stress, psi
Initial stress due to 13,000 lb pretensioning per tendon	−685	+2670	−417	+1860
Stress due to dead load on 70-ft span				
3.5 ft from center line of bearing	+160	−132	+160	−132
7.0 ft from center line of bearing	+304	−250	+304	−250
35.0 ft from center line of bearing	+845	−695	+845	−695
Combined stress due to pretensioning plus dead load				
3.5 ft from center line of bearing	−525	+2538	−257	+1728
7.0 ft from center line of bearing	−381	+2420	−113	+1610
35.0 ft from center line of bearing	+160	+1975	+423	+1165

top fibers but, since the addition of such tendons will increase the ultimate resisting moment of the section an insignificant amount, this procedure is not economical or recommended. Elastic stresses due to prestressing as well as the ultimate bending strength of the member, could also be increased by substituting tendons of greater strength for those indicated. If this were done however, increased tendon spacing might be needed to ensure that bond stresses are not excessive and that a bond failure would not occur as a result of fatigue.

Bond prevention techniques

Three methods of preventing bond have been used domestically. The first of these consists of applying heavy grease to the tendons in the areas where bond is not desired; the second method is to tape split paper tubes or heavy paper to the tendons; and the third method consists of placing split plastic tubing over the tendons in the areas where bond is to be prevented. The use of grease is considered poor practice and should not be permitted since it could be accidentally applied to the wrong tendons and escape detection. Although paper tubes or heavy paper taped to the tendons are certainly effective in preventing bond, the methods are labor consuming and are not considered as efficient or economical as the use of split plastic tubes. Excellent bond prevention has been obtained using low-cost, commercially available split plastic tubing, which is made in such manner that it clings to the tendons tightly, has sufficient overlap at the split to prevent mortar leakage, and can be installed after the tendons have been stressed at a small labor cost.

Although this method has been used successfully on at least 100 bridges and on a number of buildings in the United States alone, it has not been used as extensively as it could be for two reasons. The first and most important of these is that in early 1957 the Bureau of Public Roads issued instructions to their regional offices and to the various state highway departments that this procedure would not be permitted on projects involving federal funds. No explanation was given for this ruling. In addition, at about the same time, a New York firm advised many producers of prestressed concrete that this procedure was patented and that licenses were being granted for its use.

Some engineers believe that the use of plastic tubes may result in a weakened plane which may cause the spalling of concrete in the areas where they are used. Since plastic tubes are virtually inert chemically, if nominal reinforcing is provided in the concrete cover in the vicinity of the bond prevention and if the ends of the members are adequately sealed against moisture penetration, the possibility of such spalling seems remote. No such spalling in existing structures in which this procedure has been employed is known to the author. Research

into the durability of this type of construction would be of definite value.

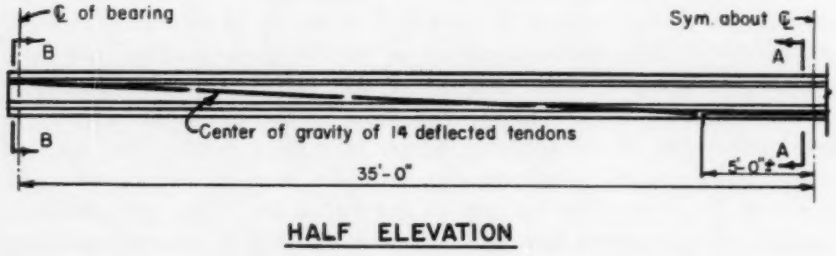
METHOD II — VARIATION OF THE PRESTRESSING MOMENT BY THE USE OF DEFLECTED PRETENSIONED TENDONS

Although the practice of deflecting tendons as a means of obtaining greater prestressing eccentricities near the center than at the ends has been fairly common practice with double-T roof slabs for several years, this technique has only relatively recently been introduced in the field of pretensioned roof and bridge girder construction, where considerably larger prestressing forces must be deflected through much greater distances. The principle and effectiveness of this method can again be illustrated by considering the AASHO-PCI Type III girder prestressed as shown in Fig. 3. Stresses resulting from an initial prestress of 13,000 lb per tendon at the center section, where all of the tendons are horizontal and in the lowest position, as well as at the end where the 14 deflected tendons are in their highest position, are summarized in Table 2. Note that the tendon layout and elastic stresses at the center of this beam are the same as for the beam which has bond prevented on 14 tendons, which of course means that each of these beams has the same capacity to resist moment (elastic and ultimate) at the center. The stresses at the end of the beam with deflected tendons are all compressive, however, and, as a result, it is not necessary to provide reinforcing in the top flange as in the previous example.

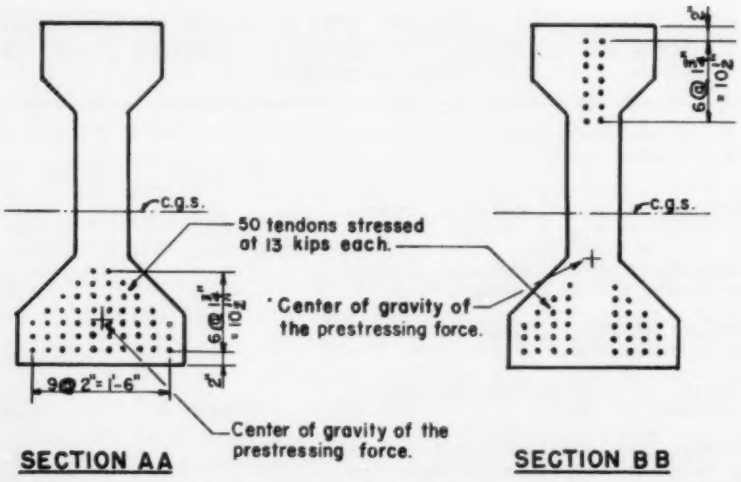
TABLE 2—STRESSES IN AASHO-PCI TYPE III BRIDGE STRINGERS WITH VARIOUS ARRANGEMENTS OF DEFLECTED TENDONS (13,000 LB PER TENDON)

Prestressing and loading conditions	Stress, psi	
	Top fiber	Bottom fiber
36 straight, 14 spaced, deflected tendons as shown in Fig. 3, Section B-B	+406	+1780
36 straight, 14 spaced, deflected tendons as shown in Fig. 3, Section A-A	−685	+2670
36 straight, 14 bundled, deflected tendons as shown in Fig. 4	−835	+2800
36 straight, 20 bundled, deflected tendons as shown in Fig. 4	−921	+3120
36 straight, 20 bundled, deflected tendons as shown in Fig. 4, less the stress due to dead load of the stringer at the center of a span of 70 ft	− 76	+2425

Note also that in the example of Fig. 3 the deflected tendons have the same vertical and horizontal spacing at their lowest and highest positions. This has been the normal practice in bridge construction when this method has been used. In the construction of double-T roof slabs it has been common practice to "bundle" the tendons near the center, which results in a greater eccentricity at the center, while the spacing of the tendons is maintained near the ends of the beam so that concrete can be adequately placed around the tendons to insure good transfer bond in the anchorage zones. This bundling is considered good practice for bridge members as well as for roof members, since the flexural bond stresses should be no greater problem with bundled pretensioned tendons than in grouted post-tensioned construction. Using this bundling procedure, Section A-A of Fig. 3 would be as shown in Fig. 4, while Section B-B would remain unchanged. Stresses for this



HALF ELEVATION


c.g.s.
50 tendons stressed at 13 kips each.
Center of gravity of the prestressing force.
9@2"=1'-6"
Center of gravity of the prestressing force.
c.g.s.

SECTION AA SECTION B B

Fig. 3 — AASHO-PCI bridge stringer, Type III, pretensioned with 14 spaced, deflected tendons

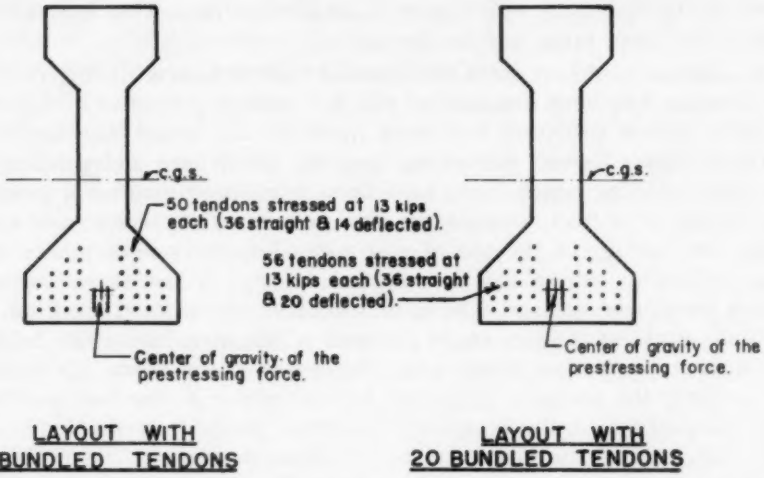

c.g.s.
50 tendons stressed at 13 kips each (36 straight & 14 deflected).
Center of gravity of the prestressing force.
LAYOUT WITH 14 BUNDLED TENDONS
c.g.s.
56 tendons stressed at 13 kips each (36 straight & 20 deflected).
Center of gravity of the prestressing force.
LAYOUT WITH 20 BUNDLED TENDONS

Fig. 4 — AASHO-PCI bridge stringer, Type III, pretensioned with bundled, deflected tendons

case at the center of the beam are also indicated in Table 2, from which it will be seen that by simply bundling the 14 deflected tendons, the bottom fiber compressive stress has been increased over 4 percent and that by using 20 bundled tendons rather than 14, the bottom fiber at the center line can be stressed to a net initial stress (prestress less the effect of the dead load of the girder) of 2425 psi which is the maximum value that could be permitted under normal conditions (assuming minimum concrete strength at time of stressing is 4000 psi) if the beam were to be used on a span of 70 ft.

Disadvantages of tendon deflection

The use of deflected tendons has been a means of controlling the stresses at the ends of beams, to which design engineers have had to resort when using pretensioned construction and where the prevention of bond and the use of post-tensioning to supplement pretensioning have been either impermissible or undesirable. Although there is an impressive number of bridge structures in existence on which this procedure has been successfully employed, particularly on the Illinois Toll Highway, certain disadvantages inherent in the deflection devices and methods deserve consideration. One of these, which is quite apparent, is the larger capital investment in prestressing facilities required to manufacture beams with deflected tendons. Most of the beams with deflected tendons have been made on pretensioning benches which were designed with provision being made for the large vertical loads which accompany deflected tendons. These benches are necessarily

much more elaborate and expensive than benches which are not designed for these large vertical loads.

In addition, until recently the manufacture of beams having deflected tendons has been limited to the few newer prestressing installations in which provision has been made in the bench for the large vertical loads. Newer deflecting devices, which are independent of the pretensioning bench, have now been introduced making it possible to efficiently deflect tendons on any pretensioning bench and overcome the limiting of the use of this method to the newer plants. One new deflecting device can be adapted to any of the three methods which have been used in tendon deflection in the past: (1) jacking the partially stressed tendons up at the ends of the members while holding them down at the low points near the center of the beam; (2) pushing (or pulling) the partially tensioned tendons down at the low points of the trajectory near the center of the beam while they are held high near the ends of the beams; and (3) holding tendons in the deflected position while they are stressed individually. In addition, this device is so designed that all of the deflected tendons can be stressed simultaneously in the deflected position (either spaced or bundled) without the friction losses and variation in stress along the tendons associated with earlier devices and procedures.

Another objection to the early methods of tendon deflection in bridge girder construction is the undesirability of the common practice of using small diameter pins or rivets over which the tendons have been bent at the points of deflection. Bending any bar or wire over relatively small radii results in secondary stresses which can be quite severe. Since the pins which hold the tendons down are near the point of maximum moment, these secondary stresses produce an area of possible fatigue failure of the tendons. A search of the literature has revealed that when fatigue tests are performed on beams with tendons such as Magnel cables which have spacers in them to keep the various wires separated, the wires frequently fail by fatigue at these spacers where secondary stresses exist.[1] This objection can be overcome by using a large radius saddle as a support for the tendons in lieu of the rivets and pins.

Furthermore, the devices which have been used in the past to hold the tendons in the "down" position, and which are referred to as "hold down devices," protrude through the soffit of the beam and the soffit form to a structural component embedded in the pretensioning bench which is capable of resisting the upward thrust of the tendons. After the concrete has been placed and cured and the tendons are to be released, these hold down devices must first be released. If they are not, and the prestressing force is released at one end with large capacity jacks as is the recommended practice, the beam nearest the re-

leasing end will tend to slide away from the releasing jacks by an amount which is primarily dependent on the length of exposed tendons between the first beam and the dead end of the bench. This movement would of course be resisted by the hold down devices if they were not released first.

Releasing the hold down devices before the stress of the bench is released has two disadvantages: (1) a large upward force is applied to the unprestressed beams at each deflection point when the hold down device is released (the author has heard reports of this force resulting in cracking of beams in the top flange at the deflection points); and (2) the hold down device must be released before the steam curing is discontinued to avoid danger of vertical cracks due to the strain changes which take place in the concrete during cooling of concrete and tendons.[2] Releasing hold down devices in an atmosphere of steam with hot forms is not generally efficient, and this has prompted a number of prestressed concrete manufacturers to release the tendons by cutting them at several locations along the bench rather than releasing all tendons simultaneously with large capacity jacks. Although careful adherence to a specific procedure of cutting of the tendons, which is determined generally by trial and error, will result in stress release without large movements of the girders, this procedure is not considered good practice since research abroad and experience in this country indicate that the length of tendon required to develop transfer bond is materially greater when the tendon is released by cutting.[3] Furthermore releasing tendons by cutting may result in crack formations at the ends of prestressed beams.[2]

A further objection to the early methods used to hold tendons in the deflected position is the cavity formed in the beam soffit by the hold down device. Experience has shown that concrete which is placed in this cavity may spall unless the cavity is patched with care. Some of the newer deflecting equipment eliminates the cavity by holding tendons in the deflected position from the top rather than through the bottom. In addition, some of the newer equipment allows the release of all tendons simultaneously with hydraulic jacks and eliminates the need of prior release of the hold down devices.

The designer of beams with deflected tendons will also find that on the longer spans, where the deflection of tendons is most advantageous, thicker webs are required to accommodate the larger number of deflected tendons which must pass through the webs near the ends of the beams. This is especially true when the tendons are spaced rather than bundled. This is a major disadvantage in the use of deflected tendons since in the longer spans flexural stresses are the primary consideration and beams with thinner webs, about 7 or 8 in., are more efficient under such conditions. Relatively thick webs are desirable for

small short span beams, where shear stresses are the primary consideration.

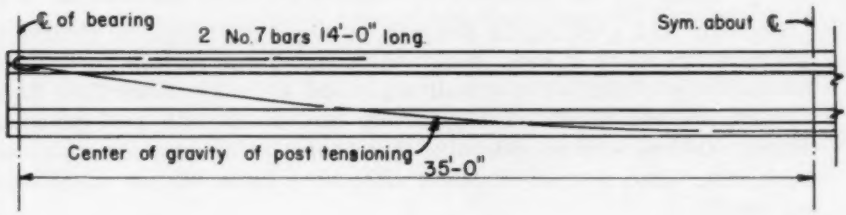
METHOD III — VARIATION OF THE PRESTRESSING MOMENT BY COMBINED PRETENSIONING AND POST-TENSIONING

Combining pretensioning and post-tensioning in a beam allows the designer to accomplish more with a given prismatic concrete section than can be accomplished with any other method utilizing pretensioning, with the one exception of deflected bundled tendons. Again the AASHO-PCI Type III bridge stringer is used as a means of illustrating this method, with the prestressing details being as indicated in Fig. 5, where the section is shown with 48 pretensioning tendons and one post-tensioning tendon of 59,000 lb initial force, as well as with 48 pretensioning tendons and two post-tensioning tendons of 88,500 lb initial force each. Inspection of stresses resulting from this procedure (Table 3), reveals that the 48 tendons used result in initial tensile top fiber stresses of approximately the same magnitude found in the example where bond prevention was considered, and hence in this solution similar reinforcing of two #7 bars would be required in the top flange at each end of the beam. Use of one relatively small post-tensioned tendon results in stresses comparable to those achieved with bond prevention and with spaced, deflected tendons. The use of two medium post-tensioned tendons in lieu of the one small tendon results in an initial bottom fiber stress similar to that achieved with 56 pretensioned tendons, of which 20 were deflected and bundled. The latter condition, as previously explained, results in net stresses for a design span of 70 ft which are about the maximum value normally permitted in the bottom fibers at the center of the beam. For longer spans, more or larger post-tensioned tendons could be used to increase the prestress in the bottom flange.

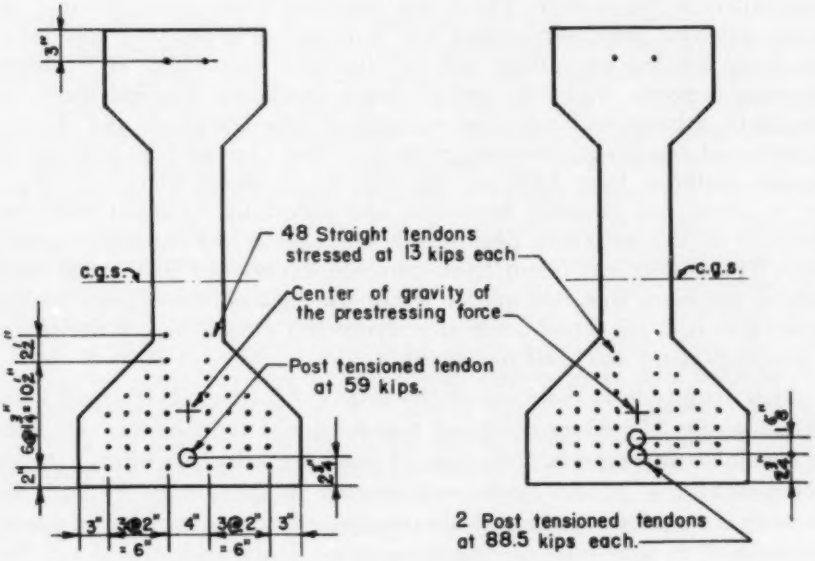
TABLE 3—STRESSES IN THE AASHO-PCI TYPE III BRIDGE STRINGER DUE TO STRAIGHT PRETENSIONED TENDONS (13,000 LB EACH) COMBINED WITH VARIOUS AMOUNTS OF POST-TENSIONING (SEE FIG. 5)

Number of pretensioning tendons	Number and size of post-tensioning tendons	Stress, psi	
		Top fiber	Bottom fiber
48	none	−390	+2350
48	One at 59,000 lb each	−487	+2620
48	Two at 88,500 lb each	−656	+3145

Note that top fiber tensile stresses and bottom fiber compressive stresses resulting from pretensioning of the 48 tendons in this example have been kept sufficiently low to permit the girder to be pretensioned and removed from the pretensioning bench before post-tensioning. This procedure is generally desirable when combined pretensioned and post-tensioned tendons are used and the members steam cured, since post-tensioning before full release of the pretensioning force calls for cooling and partial release of prestressing ahead of post-tensioning to avoid



HALF ELEVATION



ALTERNATE PRESTRESSING LAYOUTS AT THE CENTER LINE

Fig. 5 — AASHO-PCI bridge stringer, Type III, prestressed with combined pretensioning and post-tensioning

vertical cracks resulting from strain changes in concrete and pretensioning tendons during steam curing and cooling. If the pretensioning production time lost as a result of partial release of pretensioning and cooling is not serious, or if bond prevention can be used to control initial stresses at the ends of girders, the 48 pretensioning tendons required in this solution could be reduced to 42 without significant reduction of the elastic and ultimate moments which the member could safely withstand. If the post-tensioning tendons were stressed before complete release of the pretensioning force, the two #7 bars used in the top flange at the ends of the member could be eliminated.

Stressing and grouting of the post-tensioning tendons in girders of this type can serve to keep workmen occupied during lulls in the production cycle if the work is well organized and supervised. This helps to minimize the higher cost generally associated with post-tensioned tendons. Of course it is preferable that beams do not require additional work other than normal finishing after they come off the pretensioning bench.

Although some engineers rate the grouting operation as a significant disadvantage to the use of post-tensioning, this has not been the usual experience of the author. There has been difficulty with freezing of grout and with improper mixing and injecting of grout on a few jobs, but these are the exceptions and not the rule. Successful and proper grouting depends on a number of items, including the efficiency of the grout mixing and injecting equipment, the adequacy and watertightness of the grout injection ports, the care exercised in preventing foreign material from entering the sheaths or ducts before attempting to grout, the grouting technique and procedure followed, and the integrity of the workmen performing the mixing and injecting operations. The author has found from personal experience that grout containing not more than 4½ gal. of water per sack of cement can be injected into post-tensioned ducts effectively and at low cost, if the work is properly done and well organized.

Minor longitudinal cracking of the soffits of beams prestressed with a combination of pretensioned and post-tensioned tendons has occurred on a number of projects. This type of cracking, which is often not detected until the girders have been erected, is frequently difficult to see with the unaided eye and would completely escape detection in many instances if it were not for the formation of efflorescence along the cracks. The cracks are normally intermittent and occur longitudinally in the approximate center of the beam soffit. The cause of the cracks is not known at this time, although it is definitely known that freezing of the grout in the post-tensioning ducts is not the sole cause. The author has personally observed this type of cracking on five projects

using prestressed beams produced under constant supervision of the owners' inspectors, by four different prestressed concrete manufacturers. This indicates that the cracking is not due to poor workmanship on the part of a single manufacturer. The author has also received reports of this type of cracking on three other projects where beams were entirely post-tensioned. Although the soffit cracks are not normally considered dangerous since they occur at right angles to the direction of the primary stresses in the beam, all cracking is undesirable and the current lack of understanding of and ability to control this phenomenon will certainly deter use of this type of prestressing to some degree. Research into the cause of the phenomenon is obviously needed.

A further objection to use of combined post-tensioning and pretensioning from the fabricator's viewpoint is that additional equipment is required for post-tensioning and grouting, as well as for the different types of tendons, sheath, and anchorages.

COMPARISON OF THE THREE METHODS

Careful comparison of the three methods described will certainly show that bond prevention is preferable to the use of deflected tendons or post-tensioning for the example considered, if the required amount of prestress at the center of the beam can be obtained with this method. The few feet of plastic tubing and small amount of mild steel required in this method of construction can be supplied and installed at less cost than the combined cost of accessories, labor, extra capital investment, and production time required with the other two methods.

If plastic tubes as a means of bond prevention are not allowed, a choice must be made between the use of deflected tendons and combined pretensioning and post-tensioning. If a greater capacity is required of a specific concrete section than can be obtained through the use of bond prevention or spaced, deflected tendons, then the choice must be made between bundled, deflected tendons and combined pretensioning and post-tensioning. Newer deflecting equipment has made the use of deflected tendons more practical and efficient, and has eliminated the several objectionable features of friction losses, complicated stressing and releasing procedures, and stress concentrations in the tendons, as well as the spalling of soffit patches which are characteristic of the older methods. In addition, a comparison of the maximum prestressing stresses which can be obtained with 56 tendons, of which 20 are deflected and bundled, with those which can be obtained with 48 straight pretensioned tendons plus two medium sized post-tensioned tendons, would certainly indicate that the deflected tendons were more economical from purely the labor and materials standpoint. If, how-

TABLE 4—NET CONCRETE STRESSES (PRESTRESSING + DEAD LOAD) AT CENTER OF AASHO-PCI TYPE III BRIDGE STRINGER ON SPAN OF 70 FT

Type and number of prestressing tendons	Stress, psi	
	Top fiber	Bottom fiber
36 straight and 20 deflected, bundled pretensioned tendons (Fig. 4)	— 76	+2425
48 straight pretensioned tendons and two post-tensioned tendons (88,500 lb each) (Fig. 5)	+189	+2450

ever, it is feasible under the specific conditions at hand to stress the post-tensioning tendons prior to releasing the pretensioning force, or to use bond prevention on the pretensioned tendons of a combined pretensioned and post-tensioned beam, the use of bundled, deflected tendons may not be more economical from a labor and materials standpoint. The number of girders required, and the proximity of pretensioning plants which have fully or partially amortized the larger capital investment required to deflect pretensioning tendons according to best current practice, will have significant influence on the choice between these two methods for a given project.

It is significant (Table 4) that the tensile stress resulting in the top fibers of the example stringer is greater with the 20 bundled, deflected tendons than with 48 pretensioned tendons combined with two post-tensioned tendons of 88,599 lb each. In longer spans, and particularly where depth of construction is limited, the capacity of members with narrow top flanges, such as the AASHO-PCI bridge beams, is frequently controlled by the net compressive stresses in the top flange under full design load. In such a case it is desirable to obtain higher top fiber tensile stresses from the prestressing. Sometimes this is essential if the stresses are to conform to the design criteria. Similar top fiber stresses due to prestressing could be obtained with combined pretensioned and post-tensioned construction if the post-tensioned tendons were stressed before the pretensioning force was released or if bond prevention could be used to control the initial stresses due to pretensioning. Highest top fiber tensile stresses due to prestressing can normally be obtained through the use of post-tensioning alone.

If a span greater than that which can be efficiently handled with the stresses resulting from the 36 straight and 20 deflected and bundled pretensioned tendons were required, it would probably be necessary to increase the web thickness to 8 in. if bundled, deflected tendons were to be used, since such an increase in web thickness would permit three vertical rows of deflected tendons rather than the two

obtained with a web thickness of 7 in. Such an increase in web thickness would not be necessary in the case of the post-tensioned alternative where only one vertical row of post-tensioned tendons is feasible.

A comparison of the maximum vertical components of the prestressing forces, which reduce the shear force on the concrete section at the ends of the beam, in the solution with 36 straight and 20 deflected, bundled tendons stressed to 13,000 lb each, and the solution with 48 pretensioned tendons and two post-tensioned tendons stressed initially to 88,500 lb each, will reveal the vertical component to be about 23,000 lb in the case of the deflected tendons and about 28,000 lb in the case of the post-tensioned tendons. Hence, the post-tensioned tendons result in a greater reduction in the shear force carried by the concrete section, in spite of the post-tensioning force being smaller than the force in the 20 deflected tendons. This is due to the fact that the angle which the parabolic post-tensioned tendons make at the ends of the beam is substantially greater than that obtained with the deflected tendons.

It should also be pointed out that combined pretensioning and post-tensioning has been used successfully without end blocks by terminating the post-tensioning tendons at the top of the beams near the end rather than at or near middepth at the ends of the beam. The large top flange is thereby used as an end block, and has been used domestically on about 500 beams without cracking in the vicinity of the anchorages of the post-tensioning tendons. Experience has shown that this procedure can be used without cracking in the ends of the beams whereas cracking due to the concentration of stress at the ends is not uncommon in beams which have end blocks and which are prestressed with deflected tendons or with combined pretensioning and post-tensioning. The author's experience would indicate that end blocks are definitely undesirable in pretensioned construction due to the minor cracks which result at the ends when they are used.

It must be pointed out that the patent claims for the bond prevention technique and the post-tensioning methods, even though the validity of the patents may be questionable, have deterred the use of these methods and have prompted engineers and manufacturers of prestressed concrete to develop the tendon deflection method to avoid the use of these patented methods. It appears the use of deflected tendons will increase with the use of bundled deflected tendons and the improved equipment and procedures which are being developed.

CONCLUSIONS

1. Use of the bond prevention method, with split plastic tubes as the bond preventing medium, will result in the most economical means of prestressing a specific concrete section if the required strength can be developed with spaced, pretensioned tendons.

2. Virtually identical flexural capacity can be obtained in large I-beams through the use of straight pretensioned tendons, on which bond prevention is used to control the stresses at the ends of the member, and with spaced, deflected tendons.

3. A specific concrete section which is prestressed with combined pretensioned and post-tensioned tendons, or with pretensioned tendons of which a portion are deflected and bundled, will be stronger than if the section were prestressed with the same number of spaced, deflected tendons or straight tendons utilizing the bond prevention technique.

4. The use of pretensioned tendons, of which a portion are bundled and deflected, is a more economical means of prestressing a specific concrete section than is the use of combined pretensioned and post-tensioned tendons in which the post-tensioning is done after the pretensioning force is released and on which bond prevention cannot be used. The reduction in the number of straight pretensioning tendons required in combined pretensioned and post-tensioned designs in which bond prevention can be employed, or in which post-tensioning can be done economically before the pretensioning force is released, may result in greater economy than is possible with bundled, deflected pretensioned tendons.

5. Research on durability of prestressed concrete made by the bond prevention method is needed.

6. Research into the cause of cracking in end blocks of pretensioned members is needed to determine whether means can be found to prevent such cracking. Conditions under which the presence of end blocks results in cracking should be determined by systematic research.

7. Study of the cause of and means of controlling soffit cracking in beams which are prestressed by combined pretensioned and post-tensioned tendons is needed.

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Received by the Institute Mar. 25, 1959. Title No. 56-26 is part of copyrighted Journal of the American Concrete Institute, V. 31, No. 5, Nov. 1959 (Proceedings V. 56). Separate prints are available at 60 cents each.

American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by February 1, 1960, for publication in the June 1960 JOURNAL.

Title No. 56-27

Prestressed Concrete Shell for Grandstand Roofs

By HENRY M. LAYNE and T. Y. LIN

Discusses in general terms the design, construction, performance, and economy of cantilevered prestressed thin shells which form the roof over the three main grandstands of Venezuela's new National Race Track in Caracas. The prestressed shells, which constitute the roof, are 100 ft above the ground floor and extend 90 ft from their supporting columns.

The design of the roof, after first attempting a rigorous mathematical analysis, was based on the assumption that prestressing had transformed the shells into horizontal columns with no bending. Prestressing practically eliminated cracks and deflections and reduced the shell thickness to 3 in. Further information on the performance and design are given along with the method and manner of prestressing and a comparative cost estimate between the design and one using only ordinary reinforcement.

■ THE NEW HIPODROMO NACIONAL (National Race Track) now under construction in Caracas, Venezuela, is composed of many buildings and structures required for the complete operation and maintenance of what eventually will become the world's largest and most luxurious horse racing establishment. The site of the project is located approximately 5 miles from Caracas in beautiful La Rinconada Valley overlooked by the Pan American Highway.

The immediate concern of this paper is the prestressed concrete thin shell cantilever roofs over the three large five-story grandstands (Fig. 1). The main floor dimensions of the three stands designated *A*, *B*, and *C* are approximately 250 by 890 ft (Fig. 2). The prestressed roof is about 100 ft above the ground floor and covers the seating areas (Fig. 3). The total roof area is 162,400 sq ft, divided among the three stands as follows: *A* — 65,000 sq ft; *B* — 27,600 sq ft; *C* — 69,800 sq ft. Stand *C* has both a low and high roof structure. The main cantilever extends 90 ft from supporting columns. The anchor span is 52 ft 6 in. (Fig. 4). The maximum height of the cantilever from the lower point of the shell to the high point of the edge beam is 11 ft 6 in.

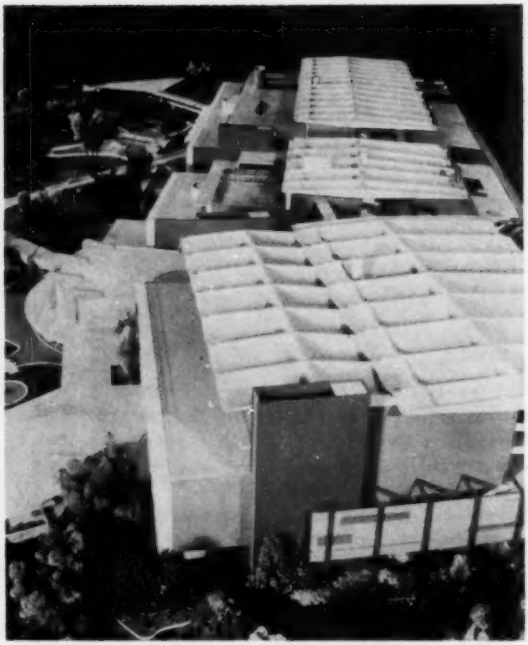

Fig. 1. — Bird's-eye view of grandstand model

The desirability of prestressed concrete instead of regular concrete reinforcement for the roofs of the stands was discussed in early planning. The architect's preliminary design called for a cantilever roof with thin scalloped edges. To satisfy the architectural design, preliminary structural considerations pointed to prestressed concrete as the answer to a lighter, sounder, crack-free roof. Since neither the ACI nor the Venezuelan codes covered the design of prestressed concrete, it was agreed to follow the most recent criteria available.

STRUCTURAL DESIGN AND DETAILS

The layout and form of the shells were chosen from both an architect's and engineer's point of view. It is interesting to note that there is practically no disagreement between the two, since the esthetically desirable shape called for in the architectural design is also structurally sound.

HENRY M. LAYNE, ACI member and organizer of its Southern California Chapter, is a structural engineer with his own consulting firm whose organization he initiated in 1945 in Los Angeles. His firm, which specializes in the structural design of buildings, was retained in 1956 to prepare the design and drawings for the Hipodromo Nacional. A portion of his structural engineering experience was acquired in the offices of Holmes and Narver, and Paul E. Jeffers over a period of 8 years.

ACI member T. Y. LIN, in addition to teaching and research at the University of California which he joined as a faculty member in 1946, is a partner in the Los Angeles firm of T. Y. Lin and Associates, consulting engineers specializing in prestressed concrete. He is a member of ACI Committees 314 and 318, and ACI-ASCE Committee 323 on Prestressed Reinforced Concrete. In 1957 he was general chairman for the World Conference on Prestressed Concrete held in San Francisco. He is the author of *Design of Prestressed Concrete Structures.*

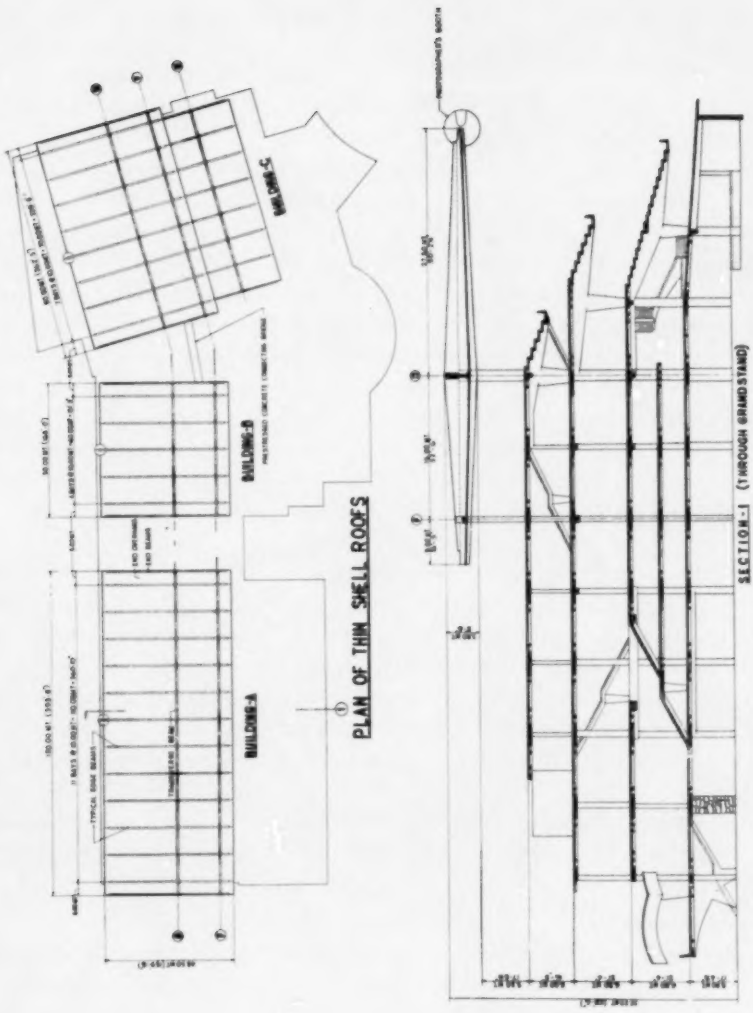

Fig. 2 — Plan and section of shell roofs

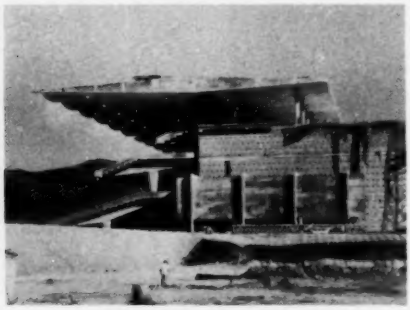

Fig. 3—Side view of shell roofs 100 ft above ground

The decision to use prestressed instead of reinforced concrete for the roof was based on the need to achieve a superior structure. It was decided that through the use of prestressed concrete, cracks and deflections could be eliminated or controlled. The consideration of these two problems is especially important in the design of suspended catenary shells and long cantilevers since excessive deflections combined with cracking could result in an unsightly structure with high maintenance costs.

The cantilever layout using a suspension shell enables a long span to be carried with a minimum of material. The inverted shell serves as the compression flange of the cantilever, while the upturned edge beam simply houses the prestressing tendons which supply the tensile force. The suspended catenary which puts shell concrete under tension is undesirable for ordinary reinforcing, but is found to be a good design when prestressed. By prestressing it is possible to reduce the shell thickness to 3 in. and the edge beams, which house the tendons, to 18 in. (Fig. 5).

A variable depth is used for both the shell and the edge beam. This also results in a sound structural layout. First, the weight near the ends of the cantilever is reduced since it produces larger bending moments. Then, a decreasing depth toward the end helps to achieve a better balance between the stresses in different portions of the cantilever and to ensure a minimum compression in the concrete. The shear force in the cantilevers under dead load is completely balanced by prestressing. Furthermore, sharper curvature of the shell near the support permits higher resistance against buckling at locations of high compression, while a flatter curvature at the end gives stiffer diaphragm action against lateral loads.

Loads and stresses

The live load for the structure as specified by the Venezuelan code was 20 psf. Since the roof curves downward, there will be no suction on top of the roof due to wind, although a suction of 10 psf is considered in the design. Downward live load, in combination with wind suction at the bottom of roof, may amount to 30 psf. This is considered the worst condition, and some tension is permitted in the concrete of the edge beams under this loading. The roof structure is designed for a

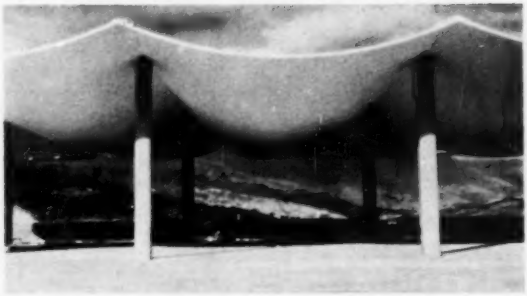

Fig. 4.—Prestressed concrete catenary shells viewed from rear of anchor span

lateral force of 6.6 percent times gravity loads, following the Venezuelan code, similar to Zone 2 of the Uniform Building Code.

For an unusual structure of this proportion it is neither safe nor economical to use the ordinary allowable concrete stresses for design. In fact, compressive stresses resulting from prestressing are rather low even in such a long span, hence they are not important except at local points of high stress concentration. The two major criteria for design, therefore, are the ultimate strength of the structure, and its proper behavior under service conditions.

Since the chances for excessive live or dead load on the roof are almost nonexistent, the only real overload might be from the wind insofar as ultimate strength is concerned. Due to the high dead load to live load ratio, its capacity to carry excessive wind load is high. The most important criterion, then, is the proper behavior of the shells.

Since the only load on the roof will be its own weight, it was deemed best to have the roof remain horizontal under the action of prestressing and dead load, with no upward or downward deflection. Thus, the cantilever shell is transformed into a horizontal column, with no shear or bending moment most of the time. From preliminary calculations it was determined that this could be achieved economically. Thus, the shell is designed for the most desirable behavior and checked for ultimate strength. Cable layout is shown in Fig. 6.

Design

The first attempt to design the shell was by a rigorous mathematical analysis. However, due to the catenary shape of the shell and the variable sag, such an analysis became too complicated. Fortunately, prestressing transformed the structure into a horizontal column with no bending at all, and shell action simply does not exist under dead load.

For live load analysis the beam theory is considered sufficiently accurate, since the ratio of span to width is about 3 to 1. Hence, the beam theory is applied to compute the ultimate as well as the elastic strength of the cantilevers.

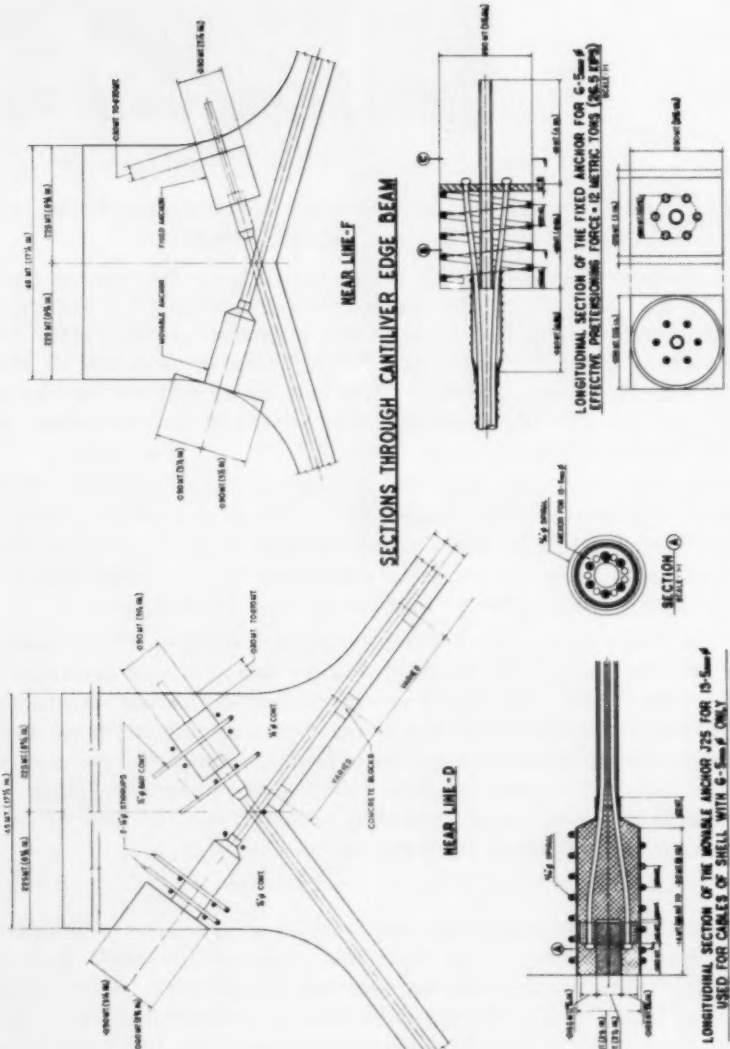
SECTIONS THROUGH CANTILIVER EDGE BEAM
NEAR LINE-F
NEAR LINE-D
LONGITUDINAL SECTION OF THE FIXED ANCHOR FOR 6-15mm ϕ
LONGITUDINAL SECTION OF THE MOVABLE ANCHOR J25 FOR 19-5mm ϕ
USED FOR CABLES OF SHELL WITH 6-5mm ϕ ONLY
SECTION
SECTION
SECTION

Fig. 5—Details of edge beams and shell cables

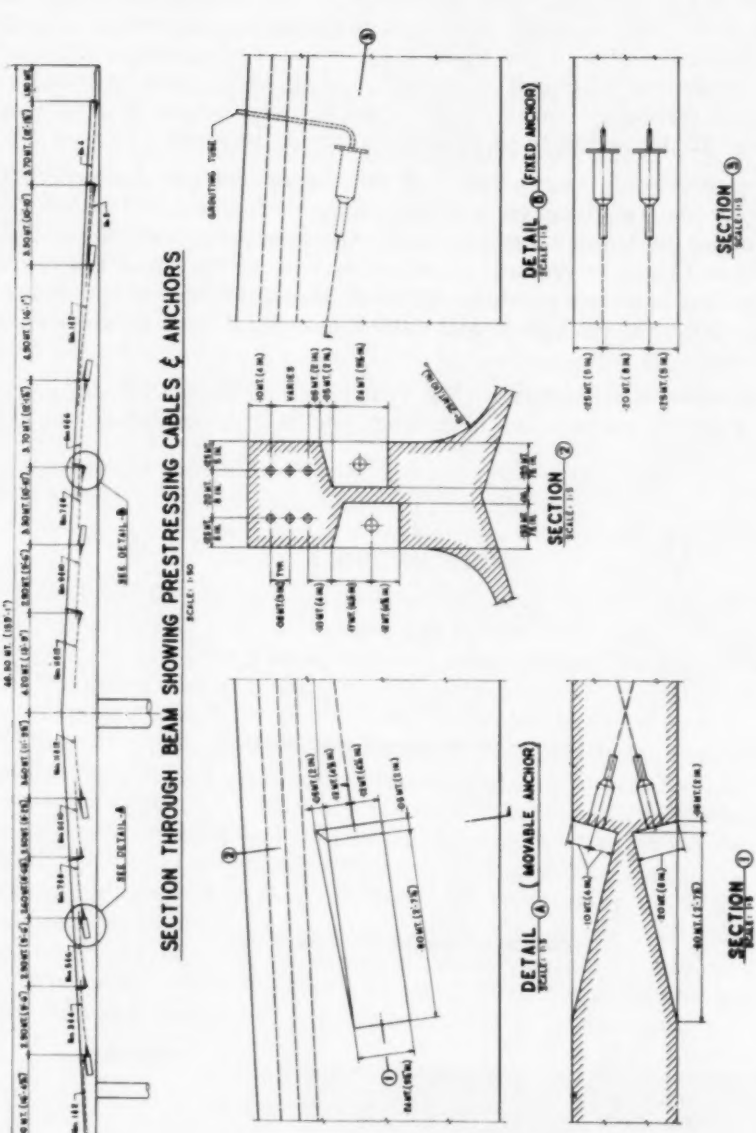

Fig. 6—Cable layout for edge beams of shell

The thickness of the shell is fixed at 3 in. to facilitate the placing of concrete and to give sufficient protection to the prestressing and reinforcing steel. Only a small amount of prestress is required to balance the suspension action of the shell, but a residual compressive stress of 250 psi is desired to ensure a crack-free shell. Mild steel is added to limit local shrinkage cracks which might develop before or after prestressing. It also serves to increase the ultimate strength.

It was previously mentioned that the cantilevers are designed primarily for the condition of dead load only so that no deflection will occur under permanent loading conditions. Actually, the initial prestress is sufficient to produce a camber of 1 in. at the tip of the cantilever, so that in time when some of the prestress is lost through shrinkage and creep, the cantilever will remain horizontal with no downward deflection.

The transverse beam design (Fig. 7) is governed by several considerations. First, it carries the load from the shell transversely to the

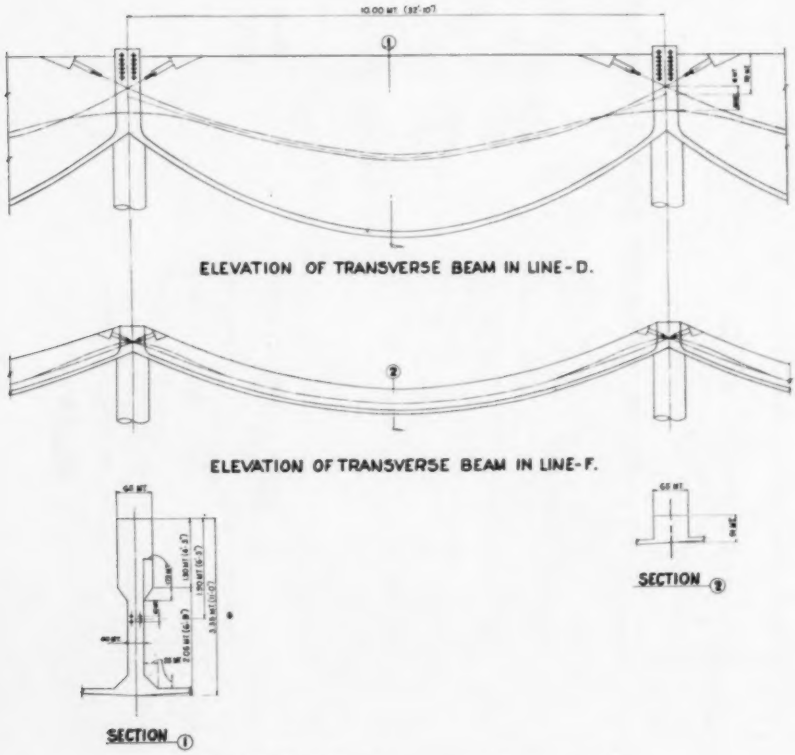

Fig. 7—Details of transverse beam cables

columns. Second, it forms a rigid frame with the columns and carries the lateral force moment. Third, it acts as a stiffening member for the shell. Fourth, its prestress is the same amount as that in the shells to minimize differential creep between it and the shells.

The anchor columns are subjected to uplift forces and are prestressed vertically to put them under compression, thus giving an uncracked effective section minimizing the deflection of the shell. They are also reinforced with steel bars to carry earthquake loads. Thus, minor horizontal forces will be resisted by the prestress with little deflection, while additional load capacity and energy absorption are provided by the steel bars.

One of the problems was the edge shell which was originally designed without an edge beam. The design was simplified by using a half shell and adding a beam at the exterior edge of the shell. These edge beams were put under the same uniform compression so they would act essentially as horizontal columns.

There are several different systems of prestressing available for this design but the general contractor submitted the BBRV system, developed in Switzerland, which was approved and adopted.

CONSTRUCTION

Forms for the shells were constructed under the entire roof area of each grandstand. The contractors decided on this method rather than forming sections for the progressive re-use of forms. The limited amount of time granted by the Ministry of Agriculture and Animal Husbandry to complete the construction greatly influenced their decisions.

Plywood forms were used throughout for the exposed concrete surfaces. The contours and support for the plywood under the shells were generally achieved by placing 1 x 6 in. boards 12 to 15 in. on center on top of 4 x 4 in. timbers resting on shaped wood trusses spaced about 5 ft apart. The wood trusses were supported on steel pipe scaffolding under the cantilevers and on wood shores below the anchor spans.

Forms for the upturned edge beams and transverse beams were supported on the plywood by means of reinforcing bar frames resting on concrete blocks. Wooden boxes, to form the openings for cable anchors and stressing equipment, were attached to the beam forms.

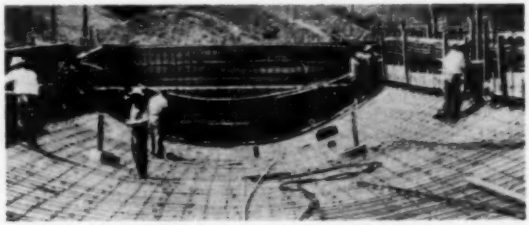

Fig. 8—Placing reinforcing steel and prestressing cables in the shell

To compensate for the anticipated shortening of the steel pipe scaffolding under the cantilevers, the forms were raised from zero to 1¼ in. higher than design elevations. Measurements of elevations made before and after concrete placement showed form settlements of 1 in. plus at ends of the cantilevers.

Placing of tendons and reinforcement

Tendons and reinforcing steel were fabricated at the site. Tendons were precisely cut to length and made into cable sizes as required. Normal procedures were followed in placing the shell reinforcement (Fig. 8). First, #3 bars were placed parallel to edge beams. Next, the "six-wire" cables in metal sheaths and #5 bars were laid in position according to their required catenary shape. Finally, the top layer of #3 bars was placed parallel to the edge beams. Steel was supported on concrete blocks.

The placement of cables and reinforcement in the beams (Fig. 9) progressed smoothly after initial handling and support problems were solved. In general bar frames, about 6 ft apart, supported the cables. The stressing end of the cables was fastened securely to the forms and surrounded by bars to take care of high stress concentrations. Stirrups and longitudinal bars were placed after all cables were in position.

During the handling of cables and placing of bars some difficulty was experienced in keeping the thin metal cable sheaths free of holes. However, before concrete was placed the sheaths were patched and cables aligned in their proper positions.

Concrete

All concrete was delivered to the job ready mixed. A minimum compressive strength of 4500 psi at 28 days was called for in the design. The concrete was required to test 4000 psi before any tensioning of cables was permitted.

The concrete mix used for Grandstand *A* was 1 part cement, 2.074 parts sand, and 2.074 parts No. 2 crushed stone; for Grandstands *B* and *C* it was 1 part cement, 1.94 parts sand, and 1.79 parts No. 2 crushed stone. The cement content was 7.64 and 8.40 sacks per cu yd, respectively, for the mixes. The water-cement ratio by weight was 0.45. Slump for both mixes averaged 3 in. An admixture was added at the rate of 4 ounces per sack of cement to retard the set and increase workability.

The required 4000 psi compressive strength of concrete for prestressing was reached within 10 days for the 7.64 sack mix and within 6 days for 8.4 sack mix. The 28-day strength of both mixes was over 4500 psi.

The concrete was lifted to roof hoppers through machines and pipes at the rate of 15 to 20 cu yd per hr. The concrete pumps were supplemented by concrete hoists or a platform hoist carrying two concrete buggies. The concrete was transported from the hoppers in hand pushed

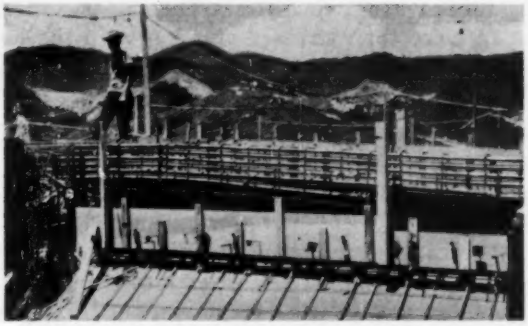

Fig. 9—Forms for edge beams with cables in place

buggies to the area of its final position in the forms. Either one or two 33 ft bays were cast continuously between construction joints located midway between columns at the low point of the shell. About 102 cu yd were required for each 33 ft bay. Internal vibrators, 1 and 2 in. in diameter, were used to compact the concrete.

Normal curing was achieved by spraying with water every few hours. A portion of Grandstand *A* was covered with burlap and sprinkled to help concrete retain moisture. The usual intermittant rain showers helped.

PRESTRESSING

The prestressing system chosen was the Swiss-developed BBRV system. This system uses straight wire of 0.20 in. diameter enclosed in thin metal conduits. Button heads formed on the wires provide the anchorage to fixed plates or stressing washers. The stressing washers, after pulling, are held in place by a nut bearing on a plate or by a similar temporary anchorage, which is removed after the cable has been grouted.

The number of wires per cable on this project varied from 6 in the shells to 42 in the transverse beam. All of the cables in the shells, columns, and F-line (anchor column line) beams were of the grouted anchorage type. The anchorages for the cables in the cantilever and transverse beams were all provided with nuts to hold the stressing washer in position after pulling.

The column cables were stressed first. Stressing of the shell cables was started next, proceeding outward from the transverse beam along the anchor span, and then, the cantilever. A second crew started on the transverse beam near the center of the building and continued to one end. This crew then finished the transverse beam working outward toward the other end. The F-line beam was then stressed in the same manner as the transverse beam. After the shells on both sides of a cantilever beam had been stressed, the cables in the cantilever were stressed starting with the live-ends in the cantilever portion. The longest

of these was stressed first. The cables with the live-ends in the anchor span were then stressed starting with the longest.

The shell cables were stressed and anchored at a wire stress of 156 ksi. The force for these cables was measured by the pressure gage on the hydraulic jack pump. The gages were calibrated every few days with an accurate mechanical dynamometer. The shell cables were grouted as soon after stressing as possible. Temporary anchor plates were removed 2 days or more after grouting.

The cantilever beam cables were initially tensioned and anchored at a wire stress of 142 ksi. After a minimum wait of 1 day, they were tensioned to a stress of 156 ksi, and finally, anchored at a stress of 142 ksi. Grout was injected after the second tensioning. For the initial stressing the pressure gages were used to determine the proper force. The dynamometer was used on each cable to determine the proper force during the second tensioning.

The water-cement grout injected into the cable enclosures at a pressure of 10 atmospheres had a water-cement ratio by weight of 0.53. An intrusion aid was added at the rate of 1½ percent of the cement weight. This aid, a compound developed especially for the BBRV system, contained a retarder and an expanding agent.

The elongation of each cable was measured at the time of stressing. The elongations showed that the average friction factor was close to the assumed value of 0.24 for the BBRV system as the oil emulsion used helped reduce the friction. The elongation check showed that although some of the cable sheaths leaked mortar at the time of placing concrete, it did not bond to the wires sufficiently to prevent the total length of the cable from being tensioned. The mortar leakage, in some cases, prevented grouting the complete enclosure around the cables.

Deflections and removal of forms

The ends of the cantilever beams, when completely stressed, deflected upward from the forms a maximum of 1.18 in. The anchor span portion of the roof raised away from the forms as much as 0.79 to 1.18 in.

The forms and shores were removed from the stressed area of the roof when a sufficient number of bays had been completed. Two stressed shells with their common stressed cantilever beam were kept supported adjacent to the uncompleted portion of the roof. Before the scaffolding was removed it was lowered and used as a platform to plaster areas where the mortar had run out of the forms and to spray a coat of paint on the underside of the roof.

Measurements made after removal of the forms showed no change in the elevation of the cantilever portion of the roof. The amplitude of vibration at the end of the cantilever beams was estimated to be less than 0.10 in. in a wind of about 20 mph.

COMPARATIVE DESIGNS AND ESTIMATE OF COST

Two complete structural designs were made for a typical bay of the cantilever roof. One, using prestressed concrete plus bar reinforcement and the second, using only ordinary reinforcement without prestress. The reason for making the second design was to examine and discuss the relative merits of each. The ordinary reinforced design had several important disadvantages, which are presented as follows:

1. The edge beam housing the tension steel had to be wider and deeper due to the much greater area of steel required to efficiently resist the moment.
2. Shell thickness increased to 3½ in. to allow more protection for reinforcing bars since many more cracks would occur.
3. The increase in weight of the roof due to the extra concrete needed would result in higher costs for earthquake resistance and foundation structures.
4. The cantilever would deflect seriously, and the amount of deflection increases with creep in the concrete.
5. The cantilever could crack noticeably at the usual design stress of 20,000 psi in steel, since the dead load is so much heavier than the design live load. Allowable steel stress should be decreased to 12,000 psi under dead load only.
6. The thin shell would be under tension due to the catenary action, and would crack noticeably. Even if the stress in the steel is kept low, cracks would still exist in the shell.

PRESTRESSED ROOFS — ESTIMATED SUMMARY OF COST*

Item	Total Cost	Cost per sq ft (162,402)
Scaffolding and shoring	$237,453.00	$1.462
Roof vault forms	169,219.00	1.042
Erect roof vault forms	169,430.00	1.043
Forming beams	119,304.00	0.735
Reinforcing steel in place	78,726.00	0.485
Concrete placing equipment	15,620.00	0.096
Concrete placing labor	42,392.00	0.261
Curing materials and labor	5,588.00	0.034
Concrete materials	53,424.00	0.329
Subtotal	891,156.00	5.487
Post-tensioning roof	185,041.00	1.139
TOTAL	$1,076,197.00	$6.626 per sq ft

Building *A*	430,480.00
Building *B*	182,954.00
Building *C*	462,763.00
	$1,076,197.00

*Note: It is estimated that the total cost of $6.626 per sq ft could be reduced to approximately $4.50 in the United States by more efficient use of scaffolding, forms, and lower cost of materials.

7. The combination of maximum moment and shear in a cantilever of this size would result in a critical condition at the support. The cracking of the beam section would reduce the shear capacity of the beam concrete, leaving the shear to be carried essentially by the shell concrete. Thus, more heavy web reinforcement would be needed than that required by code formulas.

Quantities of concrete, reinforcing, prestress wires, and forms required for both designs were computed for one bay and transverse beam of the cantilever shell. The estimated cost of these materials in place was approximately the same for each design so that the choice of design based on first cost was not critical. However, as pointed out earlier, the prestressed design is far superior in that it practically eliminates cracks and controls deflections.

A detailed cost estimate for the roof structures was made by a professional estimator employed by the architect. The unit costs for material were liberal since some adjustment was made for higher material costs in Caracas. Labor costs were based on hourly rates established for public work in the United States. The summary of the cost on p. 421 includes all structural items necessary to complete 162,402 sq ft of roof.

CONCLUSION

The successful completion and performance of the roofs for these large grandstands further point out the adaptability and potentialities of prestressed concrete shell structures, especially on long spans.

The combination of concrete and prestressing develop virtually a homogeneous material wherein the stresses can actually be adjusted to fit existing loading conditions and performance requirements.

It is important to have competent and thorough field inspection for all prestressed concrete work to ensure that the construction follows the design requirements.

ACKNOWLEDGMENTS

The Hipodromo Nacional is owned by the government of Venezuela and is administered by the Ministry of Agriculture and Animal Husbandry. The entire project was designed by Arthur Froehlich and Associates, Architects, Beverly Hills, Calif. The structural design was done in the office of Henry M. Layne, Consulting Structural Engineer, Los Angeles, Calif. T. Y. Lin and Associates, Los Angeles, were consultants on the design and supervision of the prestressed concrete shell roofs for the grandstands.

Presented at the 55th annual convention, Los Angeles, Calif., Feb. 25, 1959. Title No. 56-27 is a part of copyrighted Journal of the American Concrete Institute, V. 31, No. 5, Nov. 1959 (Proceedings V. 56). Separate prints are available at 50 cents each.

American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by February 1, 1960, for publication in the June 1960 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Stiffening effects of edge beams on a right slab bridge

J. A. N. Lee, *Engineering* (London), V. 185, No. 4807, Apr. 25, 1958, pp. 539-542

Reviewed by Aron L. Mirsky

After a brief but interesting historical résumé of the investigation of rectangular slab bridges with edge beams, author presents his method of dealing with rectangular slabs simply supported along two opposite edges and provided with edge beams along the other two edges (which are otherwise free). Shear and bending moment reactions along edge beams are expressed in terms of Fourier series, the coefficients being obtained by equating deflections and slopes of slab and edge beams. Method was experimentally verified using first sheet brass to simulate slabs without edge beams and then gypsum plaster models to simulate slabs with edge beams; agreement was quite good.

Thin arches carry Illinois toll road

Robert J. McFarlin, *Civil Engineering*, V. 29, No. 8, Aug. 1959, pp. 36-37

The Northern Illinois Toll Highway over the Fox River is spanned by a bridge consisting of five thin barrel arches. These arches, although spanning 178 ft each, are only 14 in. thick. These unusually thin arches were made possible by keeping the falsework for each barrel in place until the transverse spanning walls and roadway deck slab had been placed and cured above it. This procedure contributed to a frame action with the deck and walls helping to carry the load, thereby reducing the required thickness of the arches.

Possible subsidence countered in three new bridges

Engineering (London), V. 185, No. 4804, Apr. 4, 1958, pp. 440-441

Reviewed by Aron L. Mirsky

Three new skew bridges, two of prestressed concrete and one of steel, on a new double-track railway in a coal mining area, incorporate jacking points so that any possible future settlement can be corrected. This was necessitated both by limited headroom and by aesthetic considerations (proximity to an old church in one case).

Construction

Prince Alfred Park ice skating rink

H. F. Hespe, *Constructional Review* (Sydney), V. 32, No. 6, June 1959, pp. 19-23

Describes design and construction of an open air ice skating rink built in Sydney, Australia. This rink, being built in the southern hemisphere, required careful design of the concrete slabs due to the severe service conditions placed on it.

As corrosion of pipes and disintegration of concrete, caused by absorption of moisture, seemed to be a major fault of previously designed rinks, great care was taken to prevent this by providing two cement vapor barriers and

A part of copyrighted Journal of the American Concrete Institute, V. 31, No. 5, Nov. 1959, *Proceedings* V. 56. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

using asbestos cement chairs to support the pipes.

Another point of interest was the use of foam concrete around pipe headers and for the refrigerated slab to reduce heat transfer beneath and around the slab.

Concrete arches support a Pittsburgh apartment building

R. M. Gensert and Emil C. Hach, *Civil Engineering*, V. 29, No. 6, June 1959, pp. 47-49

Describes the design and construction of a Pittsburgh apartment building which is supported by ten reinforced concrete arches. These arches are 15 ft high and are 32 ft long (at the top) and support a two-column, bent for the full height of the building. The design analysis was based on continuous-frame analogy and employed a model for comparison of design values.

Shoreham Harbour development

Harry Ridehalgh, *Proceedings*, Institution of Civil Engineers (London), V. 11 (session 1958-59), Nov. 1958, pp. 285-296, discussion, June 1959, pp. 218-233

Reviewed by Aron L. Mirsky

Harbor development was required to facilitate entry of colliers destined for old (1906) and new (1954, 1955-58) Brighton power stations. Breakwaters were constructed of mass concrete placed between two rows of permanent steel sheet-piling. Some trouble was encountered in producing a satisfactory mix for this. Lock construction is also described.

The discussion, covering many of the problems of design and construction encountered in this project, adds much to the value of the paper.

Construction of coal mixing bins for the Bhilai steel plant

N. R. Nagaraja Rao, *Indian Concrete Journal* (Bombay), V. 33, No. 5, May 1959, pp. 151-156

Coal for the Bhilai steel plant is brought from three deposits, crushed in the crushing plant, stored in the mixing bins, and then discharged in given proportions to the final crushing plant. The mixing bins consist of ten interconnected circular silos in one row. Each silo has a diameter of 10.00 m, a storage height of about 18 m including the hopper, and is capable of storing nearly 7600 tons of coal. Originally the bins were designed to have reinforced concrete walls and steel hoppers. Later it was decided to construct the hoppers also in reinforced concrete. This article discusses the design features, and the steps in construction. The superstructure is divided into four parts: walls up to 10 m; walls above 10 m; the roof slab; and the hopper. Diagrams show reinforcement of various sections and plans for formwork and centering.

Faster work with concrete tunnel linings

Engineering (London), V. 185, No. 4798, Feb. 21, 1958, pp. 248-249

Reviewed by Aron Mirsky

Brief description of tunneling and lining method used for construction of Potters Bar railway tunnel. Tunnel, 12 miles north of London, is in clay, and is lined with 31 ft O.D. concrete rings formed of unreinforced (except in invert) interlocking segments, placed with dry joints and without grouting. Method seems to have possibilities for lining long-proposed channel tunnel.

Project studies for the reconstruction of a quay in the wet docks of the Port of Bordeaux (Les etudes du projet de reconstruction d'un quai dans les bassins a flot du port de Bordeaux)

R. Boeuf, *Annales des Ponts et Chaussees* (Paris), V. 129, No. 2, Mar.-Apr. 1959, pp. 125-140

Reviewed by Aron L. Mirsky

South quay of wet dock No. 1 collapsed over its entire length in 1943, due to an ill-timed emptying of the basin. Reconstruction was finally started in 1955, using a design which provided for a concrete wharf on reinforced concrete piles. Major problem involved the foundation material; ar-

ticle describes the successive designs, 1946-1955, which ranged from massive two-column bents, to the multiple-pile design finally adopted, with the piles constructed through new banks of rock-stabilized gravel.

Construction Techniques

Power buggies in building construction

Building Construction Illustrated, V. 28, No. 1, July 1959, pp. 51-53

Tabulates information for power buggies (up to 1 cu yd capacity) by eight different manufacturing companies. Information is given by model number with regard to dimensions, capacities, weight, horsepower, speed, and other operating items.

Surface finish of concrete

J. G. Wilson, *Constructional Review* (Sydney), V. 32, No. 6, June 1959, pp. 24-30

Discusses various means for attaining desired architectural treatment for surface finishes of concrete, and importance of formwork details, suitable concrete, and required reinforcement. Then, describes various types of surface finish for desired effect; board-marked finishes, finished obtained by use of special formwork linings, exposed aggregate finishes, applied finishes, and precast slabs.

A new system of sliding shuttering

Indian Concrete Journal (Bombay), V. 33, No. 5, May 1959, pp. 176-177

Describes a new method of using sliding form systems for casting walls. The hoists are mounted on rack-rails (soldiers) running up the outside of the concrete wall under construction. The equipment essentially consists of 4 ft high panels representing the section of the walls to be concreted simultaneously, and the hoisting plant. The number and spacing of the hoisting units depends on the shape of the building, on the weights to be supported by the platform, and on various other factors. The spacing of the units varies from 5 to 10 ft. The standard spacing for flat wall surfaces is 8 ft. The system makes it possible to construct buildings of circular, polygonal, or composite cross section, the only condition being that the cross section must be more or less uniform for the whole height for which the sliding panels are to be used. As a rule, work is continued without a break (24 hr per day) so as to obtain a completely monolithic structure without joints. The rate of erection is, on an average, about 20 ft in 24 hr. This may be increased to 22 ft when climatic conditions are particularly favorable.

Dynamiting did it where drop ball was out of bounds

William F. Hallstead, *Roads and Streets*, V. 102, No. 6, June 1959, pp. 69-71

Describes the procedure of removing four reinforced concrete pylons from a bridge so as to allow for widening. As this bridge spanned the main line of a railroad and carried gas and water mains it was deemed unwise to use a wrecking ball because of the resulting damage from vibrations. Dynamiting was finally employed and by using millisecond delay caps and a 12 ft sq steel mesh blasting mat, both concussion and flying debris were reduced to a negligible amount.

Use of membranes for curing of concrete

P. J. Jagus and K. L. Sethi, Indian Roads Congress (New Delhi), *Road Research Bulletin* No. 5, 1958, pp. 49-70

Highway Research Abstracts May 1959

The study deals with the efficacy of using different membranes, both liquid and solid, for curing concrete. The various membranes tried were plastic film, waterproof packing papers, waterproof hessian, bitumen cutback, bitumen emulsion, bitumen primer, and wax emulsion. The results indicate that the solid membranes are generally superior to the liquid membranes. Out of the solid membranes, plastic film

gave the best results and thus deserves field trials. Use of this type of membrane in arid regions, where there is shortage of water, will affect economy in the cost of concrete curing.

Dams

Baroque tendencies in the evolution of the gravity dam

SERGE LELIAVSKY, *The Engineer* (London), V. 206, No. 5356, 5357: Sept. 19, 1958, pp. 452-455; Sept. 26, 1958, pp. 490-493

Reviewed by ARON L. MIRSKY

First part of paper should be most fascinating for engineers, dam and otherwise: a discussion of the meaning of "baroque" in the title. The concept of breaking away from the formalized, the traditional, the stereotyped is then used as an analogy in the development of the buttress or "hollow" type of dam. The history of this type of dam, from Ambursen's Theresa Dam of 1903 to the present, its advantages and disadvantages (including the greater vulnerability of the hollow dam to wartime damage), the stress calculations, and the latest developments are interestingly presented.

North of Scotland hydro-electric schemes, No. IX—The Breadalbane scheme

The Engineer (London), V. 206, No. 5346-5349, 1958: July 11, pp. 44-46; July 18, pp. 84-89; July 25, pp. 124-126; Aug. 1, pp. 164-165

Reviewed by ARON L. MIRSKY

Another in a series [see "Current Reviews," ACI JOURNAL, V. 28, No. 10, Apr. 1957 (*Proceedings* V. 53), p. 1016]. Project covered here is designed to yield 370 million kw-hr annually from an installed capacity of some 120 mw. Of the 5 dams, 4 are concrete buttress structures; of these, the first, Lawers, is almost identical to the Loch Sloy Dam, two other (Lubreoch and Giorra) are gravity buttresses (each representing a refinement of the previous one), and the last, Lednock, is a diamond-head buttress with an almost-vertical upstream face. Fly ash was used in the last three for economy and favorable properties in view of the aggressive Scottish waters (*c.f.* Fulton, Angus A., and Marshall, William T., in *Proceedings*, Institution of Civil Engineers, Nov. 1956, pp. 714-730; "Current Reviews," ACI JOURNAL, V. 29, No. 12 June 1957, (*Proceedings*, V. 53), p. 1170). Some unusual tunneling and tunnel lining methods were also used.

The Garry and Moriston hydro-electric schemes

CYRIL M. ROBERTS, EDGAR B. WILSON, JOHN H. THORNTON, and HENRY HEADLAND, *Proceedings*, Institution of Civil Engineers (London), V. 11 (session 1957-58), Sept. 1958, pp. 41-68

Reviewed by ARON L. MIRSKY

Description of the engineering features of the Garry and Moriston schemes of the North of Scotland Hydro-Electric Board (see also *The Engineer*, V. 202, 1956, pp. 468-470, 503-507, 545-546, 577-579, 611-614, 647-649; "Current Reviews," ACI JOURNAL, V. 29, No. 9, Apr. 1957 (*Proceedings* V. 53), with emphasis on, among other things, the upstream reinforced concrete membrane used on the Quoich rockfill dam, the use of Trief concrete on the Cluanie and Loyne dams and the use of precast concrete facing panels instead of forms in dam construction.

Provisions made for preservation of the famed Scottish salmon are also described.

French techniques in the design and construction of large dams

Travaux (Paris), V. 42, No. 286, Aug. 1958, 248 pp.

A special issue of *Travaux*, produced on the occasion of the Sixth International Congress on Large Dams, illustrates and describes the essential principles of French technique in the design and construction of large dams.

Some typical examples of such projects have been chosen from among those which have been completed in recent years or are now in the course of construction. Altogether 25 projects are described — located in France, Algeria, Ivory Coast, French Guinea, Colombia,

Venezuela, Iraq, Iran, and Turkey. This comprehensive collection covers details of planning, design, construction, materials, etc. The issue should be recommended reading for anyone interested in hydroelectric developments.

Design

Analysis of beams in space for staircases

MARTIN SCHULZ, *Indian Concrete Journal* (Bombay), V. 33, No. 5, May 1959, pp. 172-175

Methods of analysis of beams in space when used for staircases are discussed. Considers displacement in common forms of application then develops formulas for helicoidal staircase with full uniform load.

Bending-free membranes

J. PFLIKAN, Thesis for doctor of science, 1957, 29 pp.

HUNGARIAN TECHNICAL ABSTRACTS V. 10, No. 4, 1958

Reports on a bending-free thin shell structure that may be applied over any irregular perimeter in a plane or in space. The shape corresponds to the stress distribution and resembles a film spanning a perimeter and loaded to overpressure. They may be designed with free edges, internal ribbing, supported at points, etc.

Structural concepts in the construction of Building E 100 of the Badischen Analin- und Soda-Fabrik, Ludwigshafen (Konstruktionsgrundsatze beim Bau des Hochhauses E 100 der BASF Ludwigshafen)

HERMANN BAY, *Der Bauingenieur* (Berlin), V. 33, No. 12, Dec. 1958, pp. 445-455

Reviewed by ARON L. MIRSKY

The new BASF administration building, a rectangular skyscraper, is 110.27 m tall, from bottom of foundation slab to top of penthouse roof; of this height, 100 m are above ground, the remainder being a three-level basement (foundation) structure. Building is of reinforced concrete frame type, with an inner core housing stairs, utilities, etc. Article covers structural analysis, with particular attention to the shear reinforcement of the foundation unit; the stresses in the longitudinal core wall; design of walls, slabs., etc., for compatible creep and shrinkage; and prestressing, particularly of the roof slabs over the entrances.

Contribution to the stability of cantilever girders (Beitrag zur Stabilitat des Kragtragers)

W. STEINBACH, *Der Bauingenieur* (Berlin), V. 33, No. 11, Nov. 1958, pp. 414-419

Reviewed by ARON L. MIRSKY

Author develops expressions for determination of the critical load for a cantilever loaded at the free end with a transverse concentrated load normal to the axis and a concentrated axial load. Study is restricted to prismatic beams with deep, narrow cross sections. ($I_x \geq I_y$) and with loads applied through the centroid. It is also assumed that the loads remain parallel to their original orientations as the beam deflects.

The governing differential equation is solved with the aid of power series and the pertinent boundary conditions applied to obtain solutions for five special cases: (1) pure normal loading, pure twist (lateral buckling); (2) pure axial loading, pure buckling; (3) twisting with axial loading; (4) the general case, an oblique single load; and (5) buckling with normal (transverse) loading. It is indicated that even with the power series truncated, error is in range 0 - 4 percent. Two numerical examples are given.

Bending of conical shells of uniform or nonuniform thickness

TROELS BRONDUM-NIELSEN, *Meddelelse* No. 10, Laboratoriet for Bygningsteknik, Copenhagen, 1959, pp. 26-39

Simple, explicit formulas were derived for the elasticity of the edges of conical shells of uniform thickness and of cylindrical shells of uniform or linearly varying thickness under the action of axially symmetrical loads. Diagrams are presented for the coefficients appearing in these equations.

It was found that conventional methods involved considerable inaccuracies. The present method offers increased accuracy and simplifies the calculations.

Moment-and normal-force-carrying capacity of circular-ring foundations for given maximum soil pressure (Aufnahmevermogen kreisringformiger Fundamente fur Moment und Normalkraft bei gegebener grosster Bodenpressung)

OTTO MULLER-MAIN, *Der Bauingenieur* (Berlin), V. 33, No. 11, Nov. 1958, pp. 419-420

Tables for selection of outer and inner radii of ring foundations, given vertical load, overturning moment, and maximum allowable soil pressure. A numerical example illustrates the procedure.

Total tensile force on openings in a biaxial compression-stress field (Die totale Zugkraft an Offnungen in einem zweiachsigen Druckspannungsfeld)

R. HILTSCHER, B. PANT, and P. B. CHAULIA, *Der Bauingenieur* (Berlin), V. 33, No. 12, Dec. 1958, pp. 455-459

Reviewed by ARON L. MIRSKY

Previous work [*Bauingenieur*, V. 32, No. 12, Dec. 1957, pp. 470-474; "Current Reviews," ACI JOURNAL, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55), p. 742] is extended to biaxial fields for openings of rectangular and elliptical shape. Results are intended to be of assistance in design of tunnels, penstocks, and similar structures.

Stress calculations of the shear reinforcement in reinforced concrete beams of the ultimate load theory

T. GYENSO, *Epites-es Kozlekedestudo-manyi Kozlemenyek*, V. 1, No. 1-2, 1957, pp. 43-67

HUNGARIAN TECHNICAL ABSTRACTS V. 10, No. 4, 1958

Working on the basis that shear cracks are unpermissible under a working load the author's design procedure is to determine the principle tensile stresses by using the elastic theory, which specifies an upper limit, and then computing the amount of shear reinforcement needed by using the ultimate load theory. Some basic assumptions made in connection with this theory are: shear failure is a combined effect of shear and bending; external forces are in equilibrium with the internal forces acting on the compressive zone, tensile reinforcement, and shear reinforcement; concrete in tension takes no load; tensile reinforcement takes no shear; and compressive stress is equally distributed.

Limit design and plastic behavior of structures (Calcul a la rupture et plasticite des constructions)

A. R. RJANSITSYN, Editions Eyrolles, 1959, 287 pp.

Reviewed by A. W. COUTRIS

This important book is the third of a recent series of Russian texts translated into French, the other two being "Plasticity" by A. A. Ilioushine, 1956, and "Applications of the Theory of Creep" by N. K. Ahrouthounian, 1957, both published by Editions Eyrolles. These three volumes serve as an excellent indication of the high caliber of research being done by Russian scholars in these fields.

Author Rjanitsyn has divided his book into fourteen chapters totaling 287 pages (French translation). In the first chapter, the concepts of "limit load," the maximum load which a structure can support without collapse, and its corresponding stress-deformation condition, i.e., "limit state," are defined and examined in statically determinate and indeterminate systems from assumptions on the stress-strain curve of the material. Perfectly elasto-plastic materials are mainly considered and a number of examples are worked out. The second chapter is devoted to the bending of elasto-plastic beams. The influence of initial stress and continuous cyclical loading is studied. Chapter 3 takes up the limit design of continuous beams, examining the effects of support settlements, unloading, and repeated loads. In the follow-

ing chapter the topics are plastic bending out of one of the principal planes of the section, plastic bending produced by eccentric axial compressive and tensile loads, and limit design of elasto-plastic arches.

Chapter 5 considers the general theory of analysis of elasto-plastic systems, including variable and alternating loads. A classical plasticity treatment of bi- and tridimensional states of stress appear in Chapter 6. Studies on torsion follow those of Nadai with some interesting additions.

The plastic analysis of a cylindrical rod subjected to torsion and tension, a thin band under torsion and tension, bending and torsion of a thin band, a bar with a thin-walled open profile are treated in Chapter 7, including an approximate estimate of the influence of shear on plastic bending. Following this section is the plastic analysis of plates, having a variety of boundary conditions. Some of this material on rods, bands, and plates appears to be presented for the first time.

Chapter 9 is a study of nonlinear plastic deformations in line with the work of Prager.

Visco-elastic systems appear in Chapter 10, the general law of linear deformation (creep) in Chapter 11, and nonlinear deformation (nonlinear creep) of material in Chapter 12. Some of this work on creep does not appear in Ahrouthounian's book on creep.

Chapters 13 and 14 are devoted to a study of the safety of structures, the probability of structural failure, the determination of safety factors, and applications to the design of steel structures.

In general, the text, which does include elaborate mathematics, presents an extremely valuable and comprehensive treatment of its subject for engineers interested in deriving the benefit from the application of modern developments in limit design and conditions of structural safety to the designs of their own structures.

On analysis of stresses under arbitrary points inside and outside a uniformly loaded flexible circular area (Zur Berechnung der Spannungen unter beliebigen Punkten innerhalb und ausserhalb einer gleichmassig belasteten schlaffen Kreisflache)

H. Meissner, *Der Bauingenieur* (Berlin), V. 33, No. 9, Sept. 1958, pp. 342-344
Reviewed by Aron L. Mirsky

Starting with Fröhlich's equation for single concentrated loads,

$$\sigma z = \frac{\nu P}{2\pi z^2}\cos^{\nu+2}\phi$$

where ν is the "concentration factor" or "concentration index"* and making use of superposition and the simplifying effects of symmetry, author derives, for integer values of ν, solutions for σ_z, the vertical stress component at any point, in the form of closed algebraic expressions. Such solutions are given in both formula and diagram (curve) form for $\nu = 2$ and $\nu = 4$, cases that are important in tank design.

*c.f. Terzaghi, Karl, *Theoretical Soil Mechanics*, John Wiley and Sons, New York, 1943, pp. 395-396.

Limit design for structural concrete

Alan H. Mattock, *Journal*, Research and Development Laboratories, Portland Cement Association, Chicago, V. 1, No. 2, May 1959, pp. 14-24
Author's Summary

Experimental evidence of the inelastic behavior of reinforced and prestressed concrete structures is reviewed. Various methods of limit design, based on recognition of this inelastic behavior, are discussed. Possible methods of introducing limit design into structural concrete design practice, and the advantages to be gained therefrom are considered.

Analysis of continuous beam bridges. V. 1, Carry-over procedure

Jan J. Tuma, Samuel E. French, Jr., and Therman I. Lassley, *Research Publication* No. 3, Oklahoma State University, 1959, 67 pp.

Presents the numerical carry-over moment method to the analysis of continuous beam bridges subjected to

stationary or moving loads, applied couples, displacement of supports or change in temperature. The carry-over moment procedure is a numerical successive approximation, which may be carried out to a desired degree of accuracy. This study is restricted to coplanar systems and the customary assumptions of beam analysis are introduced. Derives the general three-moment equations for beams of variable section and the geometry and integral functions of beams with parabolic haunches. The algebraic formulas of beam constants are given for beams with one parabolic haunch and beams with two parabolic haunches and then tabulates values. Uses two examples to illustrate the analysis of continuous beam bridges by means of the numerical carry-over procedure and the application of the numerical coefficient derived and tabulated in the report. Four thousand beam constants are tabulated in this report.

On the analysis of long cylindrical shells (Zur Berechnung langer Zylinderschalen)

G. KIRCHNER, *Der Bauingenieur* (Berlin), V. 33, No. 9, Sept. 1958, pp. 331-336

Reviewed by ARON L. MIRSKY

A study of methods of analysis of long cylindrical shells, resulting in an appreciation of Lundgren's method, its power, beauty, and usefulness. Study was triggered by criticism of Lundgren's method at the second symposium on concrete shell roof construction (Oslo, 1957).

In discussing the paper, G. Röntsch demonstrates that single shells of the same basic dimensions but of different configurations can yield quite different results, and emphasizes that the validity of the Lundgren method is dependent on lack of distortion of the cross section of the shell.* In his reply, author emphasizes the automatic controls inherent in the Lundgren method, and discusses five aspects, including the influence of the form of the cross section and of the longitudinal span length, and the transverse moments.

*This point is also emphasized in "Cylindrical Shell Analysis Simplified by Beam Method," by James Chinn, ACI JOURNAL, V. 30, No. 11, May 1959 (*Proceedings* V. 55), pp. 1183-1192

Materials

Bauxite ore aggregates for shielding concrete

GR. KUNSZT, *Epitoanyag*, V. 10, No. 1-2, 1958, pp. 23-27

HUNGARIAN TECHNICAL ABSTRACTS V. 10, No. 4, 1958

Describes the use of bauxite ore for shielding protection against combined neutron and x-ray radiation. Previous to using bauxite, limonite ore was used but it was found that its water-hydrate content was not high enough. The phenomena of elastic and inelastic dispersions, and absorption of neutrons is analyzed.

Design of a functional structure in or on rock

A. V. CORLETT and C. L. EMERY, *The Engineering Journal* (Montreal), V. 42, No. 4, Apr. 1959, pp. 86-92

Reviewed by ARON L. MIRSKY

A general survey of the behavior of rock as a structural material, in relation to the stresses to which it is subjected in underground workings, and some methods of preventing failures. As author points out, "civilization has reached a stage where it appears that men may again have to go underground and live as cavemen." Paper is thus of value not only for its engineering applications (see review p. 428 of paper by Hiltscher, Pant, and Chaulia), but for its general timeliness as well.

Capillary continuity or discontinuity in cement pastes

T. C. POWERS, L. E. COPELAND, and H. M. MANN, *Journal*, Research and Development Laboratories, Portland Cement Association, Chicago, V. 1, No. 2, May 1959, pp. 38-48

AUTHORS' SUMMARY

This is a study of the capillary structure of cement paste in terms of data on permeability to water. Capillaries in fresh paste are continuous but if the

specific surface area of the cement is 1800 sq cm per g or greater (Wagner), and if the water-cement ratio is not higher than 0.7 by weight, production of cement gel during the curing period will eventually segment the capillaries, forming "capillary cavities." If the specific surface area is higher, the limiting water-cement ratio is higher, and presumably, vice versa. Some practical consequences of capillary continuity or lack of it are discussed.

What to do with left-over concrete?

Modern Concrete, V. 23, No. 3, July 1959, pp. 38-40

Discusses layout and use of an aggregate reclaimer installation used to process left over concrete. A unit separates the components of mixed concrete by a scrubbing action combined with gravitational forces. The reclaimed gravel or stone goes through the machine and drops into a truck. Sand comes out in the middle and the cement sludge goes into a settling basin. A sludge pump removes any clear water that may be at the top of the settling basin. Cement sludge is pumped into a truck mixer and hauled to the dump. The sand, as well as recovered gravel, could go back into the stockpiles; gravel was cleaner when it came out of the machine than when it entered the mixer. Sand showed a gradation only slightly off from the original gradation. The price of the gravel is reduced by the process and the haul-away problem can be reduced by half.

One solution to the left-over concrete problem

Concrete Products, V. 62, No. 5, May 1959, pp. 38-41

An aggregate reclaimer has provided one ready-mixed concrete producer with a new answer to an old problem. On return to the yard with 1/2 to 3/4 cu yd of waste concrete, the truck backs up to the unit and waters the mix enough to increase the slump to 8 or 10 in. and discharges into the unit's hopper. The watered-down material splashes into a rotating drum and the cement paste and some of the fine sand is floated away. The stone and sand are carried by a screw conveyor to the discharge end of the reclaimer through a water spray to wash away any remaining paste. Then the sand and stone enter a screening area which filters out the sand and water. The stone is given additional washing and can then be transported back to regular storage.

Physicochemical nature of the action of plasticizing and air-retaining additives

V. F. Zhuravlev and V. A. Tikhonov, *Journal of Applied Chemistry* (Moscow), V. 25, 1952, pp. 1317-24 (English translation). *Chemical Abstracts*, V. 49, No. 7, 1955, p. 4963a. *American Ceramic Society*, V. 42, No. 3, Mar. 1, 1959, p. 73

Highway Research Abstracts
June 1959

Discusses the effect of surface active agent abietene tar on the electrophoretic mobility of particles in concrete suspension. This agent increases the mobility, air retention of cement and concrete pulps, and reduces water penetration and salt action in hardened concrete. Plasticizing agents such as NH_4 lignosulfonate act in a similar manner by retarding the rate of cement particle sedimentation thereby increasing the stability of the suspension.

Gap graded concrete for pavement construction

K. S. Sankaran, *Journal of the Indian Roads Congress* (New Delhi), V. 23, No. 2, Dec. 1958, pp. 293-314

Highway Research Abstracts
May 1959

Advocates a method of concrete mix proportioning in areas where the locally available sands are poorly graded for purpose of the conventional continuous grading and imported well-graded sands are too expensive for the purpose. The concrete designed by this method is termed gap graded concrete.

The investigation deals with the proportioning of concrete mixes by using

four typical sands with poor gradations, on the gap graded principle, wherein the gradation of coarse aggregate is adjusted after deliberately omitting certain particle sizes, so as to accommodate the available sand grading without particle interference. The study indicates that concrete mixes, designed in the above fashion, using the four typical "poor" sands, compare well with a "standard" continuous graded concrete mix in terms of strength properties.

Some Russian research on cement

NATHAN C. ROCKWOOD, *Rock Products*, V. 62, No. 7, July 1959, pp. 18, 100, 112

Reviews briefly some samples of a few translated Russian research papers on cement, lime, and plaster. These papers deal with the solid phase chemical reactions of portland cement, for the most part, and are of a practical nature. Some of the topics that have been investigated are the cementing properties of (gamma) dicalcium silicate, effect of mineral composition of cement on its contraction, and the surface hardness of concrete as a measure of its resistance to freezing.

More solutions to the left-over concrete problem

RICHARD S. HUHTA, *Concrete Products*, V. 62, No. 8, Aug. 1959, pp. 34-39, 53

An extensive survey of the ready-mixed concrete industry was conducted recently to determine what use was made of left-over concrete. Most producers were using their waste concrete to pave yard and storage areas, but others were making tidy profits with small miscellaneous products cast with the excess material. Lists 34 different items that producers are making with left-over concrete and has pictures of many, with information as to manufacturing costs, selling prices, and available markets. Disadvantages of mix inconsistency, labor costs, and extra time discourage some producers from developing small products. Other producers give excess concrete away to employees, civic groups, and other individuals or organizations, to better public relations.

Errors caused by partial drying of hardened portland cement during testing

K. M. ALEXANDER, *Nature*, V. 182, Mar. 28, 1959, pp. 885-886

Emphasizes the extreme magnitude of the change in strength which can be caused by partial drying of hydrated portland cement, and directs attention to the rapidity with which this change can occur. The first series of tests demonstrated that the samples tested immediately after removal from the water bath gave higher moduli of rupture than after drying. So, later samples were immersed in a water bath during testing. A mean modulus of rupture of 1960 psi and coefficient of variation of 10 percent was obtained with immersed specimens. This compares with a mean of 1010 psi and coefficient of variation of 8 percent for test performed after more than 1 min of exposure in the air of the laboratory.

Effect of heat-treatment on the constitution and mechanical properties of some hydrated aluminous cements

SAMUEL J. SCHNEIDER, *Journal*, American Ceramic Society, V. 42, No. 4, Apr. 1959, pp. 184-193

AUTHOR'S SUMMARY

The compound compositions of four aluminous cements were determined on anhydrous as well as hydrated specimens which had been heat-treated at temperature between room temperature and 1400 C. Phases were identified by x-ray diffraction and differential thermal analysis. Specimens were also tested for transverse strength, dynamic modulus of elasticity, and thermal length change. A study of the dehydration characteristics of $CaO \cdot Al_2O_3 \cdot 10H_2O$, $3CaO \cdot Al_2O_3 \cdot 6H_2O$, and $Al_2O_3 \cdot 3H_2O$ was included. The data indicated that $CaO \cdot Al_2O_3 \cdot 10H_2O$ was the primary crystalline hydrate formed in

the cements at room temperature. At 50 C., $3CaO \cdot Al_2O_3 \cdot 6H_2O$ and $Al_2O_3 \cdot 3H_2O$ were formed as by-products of the dehydration of $CaO \cdot Al_2O_3 \cdot 10H_2O$. When heated alone in an open system, $CaO \cdot Al_2O_3 \cdot 10H_2O$ did not convert to $3CaO \cdot Al_2O_3 \cdot 6H_2O$ and $Al_2O_3 \cdot 3H_2O$. A correlation between the mechanical properties and compound compositions was noted.

An investigation into the relationship between the transverse strength and the compressive strength of cement mortar

M. F. KAPLAN, *Bulletin*, No. 17, National Building Research Institute, (Pretoria), Mar. 1959, pp. 45-53

AUTHOR'S SUMMARY

An investigation has been carried out to determine whether the results of a proposed transverse test on cement mortar can be used to indicate variations in the compressive strength of cement. There was found to be correlation between transverse and compressive strength and that the proposed transverse test is capable of indicating variations greater than 150 lb per sq in. in the compressive strength of cement mortar. The results also confirm that the transverse strength of cement mortar is proportional to the two-thirds power of its compressive strength.

"Rubber" base mortar undergoing tests

Michigan Roads and Construction, V. 56, No. 15, Apr. 9, 1959, p. 9

HIGHWAY RESEARCH ABSTRACTS
June 1959

Discusses a liquid latex emulsion that is added to the concrete during the regular mixing processes and which is primarily used for thin-patching structural concrete. It is stated that the pliability and bond of concrete using this additive is greater than ordinary concrete. An exemplification of these properties is given in the form of a rigorous field test on a disintegrating bridge deck. A minimum of ½ in. of concrete was removed from this bridge and replaced with latex mortar. Even though this patch was subsequently subjected to over 100 freeze-thaw cycles it maintained good bond and durability and was judged "very satisfactory."

Plastic insert forms contraction joint, seals crack, ties slabs

LEE WORSON, *Southwest Builder and Contractor*, V. 133, No. 15, Apr. 10, 1959, pp. 118, 121

HIGHWAY RESEARCH ABSTRACTS
June 1959

This polyvinyl chloride plastic contraction joint consists of long flexible strips, the cross section having a central bulb and stops on the wings to provide ties. It is inserted into the concrete immediately after vibration and when using a wing with ¼ in. thickness it develops a tie strength of 7000 lb per ft. It is inert and impervious to jet fuel, acids, alkalies, chlorine, and has a working temperature range of −55 to 300 F. The cost of installing this joint is competitive or slightly higher.

Pavements

Continuous steel reinforcement for experimental concrete pavements

Bulletin No. 214, Highway Research Board, 1959

Analysis of special problems in continuously-reinforced concrete pavements

WILLIAM ZUK, pp. 1-21

Topics covered are presented from a theoretical analytical viewpoint and experimentally. Some of the topics studied are pavement thickness, differences in behavior of deformed bars and plain wire mesh, buckling tendencies, horizontal and vertical alignment changes, end anchorage, crack behavior under repeated loading, and reduced slab rigidity due to cracks.

A ten-year report on the Illinois continuously-reinforced pavement

J. D. LINDSAY, pp. 22-40

Describes the behavior and performance of a experimental pavement consisting of eight test sections (four being uniform 7 in. pavement and four

being uniform 8 in. pavement), ranging in length from 3500 ft to 4320 ft. The conditions investigated are: development of transverse cracks, crack width, pavement pumping, change in length (expansion joints), riding quality of surface, maintenance, economics, and recommended design for future pavements.

Ten-year report on experimental continuously-reinforced concrete pavements in New Jersey

WILLIAM VAN BREEMAN, pp. 41-75

Reports on the behavior and performance of two experimental sections of pavement each approximately 1 mile long and subjected to heavy truck traffic. The north section is 8 in. thick and the south 10 in. thick, both contain different amounts of reinforcing steel which consists of a double line of welded wire fabric.

Discusses such topics as changes in length, crack pattern, crack width, and behavior of terminal joints; and compares cost and performance of these sections with standard design.

Continuously-reinforced concrete pavement in California after eight years service

BAILEY TREMPER, pp. 76-79

Reports on the condition of an experimental continuously - reinforced concrete pavement 1 mile in length which was built in California in 1949. The conditions investigated are similar to those of the other papers.

Continuously-reinforced concrete pavement in Pennsylvania

F. C. WITKOSKI and R. K. SHAFFER, pp. 80-97

The objectives of this project are: to study the effect of pavement thickness on service performance; study the effect of subbase thickness on service performance; determine the maximum stresses in the reinforcing steel at a crack, considering varying thicknesses of pavement; effect of traffic on stresses in steel; thermal effects on steel and concrete stresses; warping; magnitude of longitudinal movement of pavements; and ultimate slab length.

First-year performance report on continuously-reinforced concrete pavements in Pennsylvania

I. J. TAYLOR and W. J. ENEY, pp. 98-113

Reports on the results of extensive mechanical and electrical instrumentation used to evaluate the performance of two continuously-reinforced concrete highways in eastern Pennsylvania.

The first of these pavements, which was closed to traffic for the entire test thereby eliminating the influence of heavy wheel loads, was tested for the effects of temperature cycling in extreme weather conditions.

In the second pavement, numerous electrical gages were installed to permit a careful investigation of the phenomena associated with the formation of transverse cracks. Variable types of reinforcement were used, but the steel was kept constant.

Vibrating-wire strain gage for long-term internal measurements in concrete

F. P. POTOCKI, *The Engineer* (London), V. 206, No. 5369, Dec. 19, 1958, pp. 964-967

Reviewed by ARON L. MIRSKY

Describes a low-cost strain gage developed at the (British) Road Research Laboratory for embedment in concrete pavement slabs, and of the tests to which it was subjected to check and evaluate its working and reliability.

Gage, consisting of single wire stretched between two steel flanges which are maintained in correct relative position during concreting by a "Perspex" (plastic) tube, which also serves as protective cover for the wire, is distinguished by (1) low production cost — about 30 shillings (approximately $4.20 at current exchange rates); (2) low over-all longitudinal stiffness, hence minor effect on measured quantities; (3) simplicity of reading data and computing results; (4) high accuracy (in addition, calibration of individual gages is usually not nec-

essary); (5) equal reliability in tension and compression; (6) reliability over long periods (hence, suitability for long-term measurement of shrinkage of concrete under field conditions, thermal expansion of concrete, static strains in slabs under thermal warping and other slowly varying phenomena, etc.); and (7) insensitivity to effects of long leads, high temperatures, or immersion in water. Accuracy is stated to be 0.5 microin. per in. or ±2 percent.

Reviewer considers this gage a notable advancement in the field, but would like to see diameter of flanges reduced, if possible.

Some "do's and don'ts" of concrete sawing

ROBERT JANOWITZ, *Roads and Streets*, V. 102, No. 6, June 1959, pp. 78-80, 82

Discusses concrete sawing which is used on highway pavements to provide contraction joints, control of random cracking, etc.

The main topics treated here are equipment selection, operation and maintenance, and blade selection. A good deal of practicable and profitable information is given.

Precast Concrete

How to cut block moisture to 10 percent . . . inexpensively!

WILLIAM J. SHORE, *Concrete Products*, V. 62, No. 8, Aug. 1959, pp. 40-43, 55

Cracking is a problem of the block industry, but is not entirely the producer's fault. There are ways he can help overcome the problem, and one of these is to manufacture a product with a moisture content so low that shrinkage will no longer be worth consideration. Autoclaving does this, but an autoclave is expensive and probably beyond the financial means of many producers.

The author has developed a simple drying system, installed at a cost of only $7000, which handles 5000 8-in. units per 24 hr. Basically, the method of drying consists of heating up the incoming air, recirculating it through the cubes of block and then exhausting the moisture-ladened air from the room.

Test results showed that the moisture content may be reduced to 30 percent in approximately 8 hr and to 20 percent in 22 hr, but that at least 48 hr were required to reach 12 percent and less. Heat cost about $7.00 per 1000, 8-in. equivalent block dried to 11 percent moisture.

Decorative precast developments — Ideas from Europe

T. B. SMITH, *Concrete Products*, V. 62, No. 5, May 1959, pp. 33-36; No. 6, June 1959, pp. 44-48

First part deals exclusively with developments in Great Britain and reviews methods of production (vibration, wet casting, and dry pressing), size of aggregate, and exposed aggregate finishes. The second part presents a roundup of precast concrete finishes being used throughout Europe.

Germany lays first precast concrete road

THOMAS C. LUCEY, *Concrete Products*, V. 62, No. 7, July 1959, pp. 44-46

Using precast concrete slabs, a three-block road section in Stuttgart was laid in 3 weeks. The slabs are connected by interlocking wedges and are about 6½ ft square.

How to lick the culls problem

Concrete Products, V. 62, No. 8, Aug. 1959, pp. 33, 55, 57

Discusses the methods used by one cost-conscious block plant superintendent to cut his culls an average of 90 per day to about 14, out of a daily production of 15,000 block. Mixing is the most important phase of the operation Each mix is timed and checked so that it is just like the one before. Mixer man periodically inspects block as they are released from the mold. Offbearer also watches for culls and returns any bad block to the mixer. Extra man is used

to pull the racks into the kiln. From the kiln, the block are handled gently and culls are still further weeded out in stacking in segregated areas and storage yard. Good housekeeping is important. Yards, machine areas, and kilns are constantly swept clean.

Prestressed Concrete

Precast, prestressed concrete trusses carry a hangar roof

Engineering News-Record, V. 163, No. 7, Aug. 13, 1959, pp. 32-36

Reports on the construction of a maintenance hangar for an English charter airline. This hangar which is 280 ft long, 110 ft deep, and 41 ft high with a clear height of 30 ft, employs concrete space trusses for the roof. These trusses are triangular in section and are 8 ft 8 in. in depth and span 105 ft, from front to rear of the building. After prestressing the bottom chords of the truss and grouting the diagonals the trusses were then lifted to supporting columns at the rear by means of jacks. At the front end of the hangar the space trusses were supported by falsework until the end frames were joined by horizontal and diagonal precast units to form a two span truss. The entire main front truss was then prestressed with two exposed parabolic cables on opposite faces of the truss that act much like the main cables of a suspension bridge. As a final step, the lateral struts running the 280 ft length of the hangar were post-tensioned to tie the entire frame together.

Concrete for prestressed concrete (Betong til spennbetong)

SVEN THAULOW, *Betongen Idag* (Oslo), No. 2, 1957, pp. 39-66

AUTHOR'S SUMMARY

Those factors affecting the strength of prestressed concrete, its tensile strength, and deformation (creep and shrinkage) are reviewed. Emphasis is placed on the influence of moist curing in developing strength. It is shown that care must be observed in employing steam curing, only moderate temperature rises to be permitted the first 6 hr. It is recommended that tensile strength be determined as split tensile strength instead of the customary flexural tensile strength.

Both creep and shrinkage are much affected by temperature and humidity, and by so many other factors that it is not possible to include them all in the computations.

Relatively little is known about the particular factors involved in proportioning high strength concretes. For prestressed concrete it is recommended that rapid hardening cement be used. The sand should be coarse, with a minimum amount of filler; the coarse aggregate should be crushed stone, with not too large a maximum size. The concrete should be vibrated and effectively moist cured.

Prestressed concrete factory at Vallabh Vidyanagar

S. M. AMIN, *Indian Concrete Journal* (Bombay), V. 33, No. 5, May 1959, pp. 157-160

One item which provides a good scope for mass production is prestressed concrete poles for electric transmission lines. The use of prestressed concrete for poles gives a product which is low in initial cost, strong and durable, and at the same time has a pleasant appearance. Article discusses design of the Vierendeel-type pole and the factory layout and equipment. Factory is at present manufacturing 44 poles per day but it is possible to increase the rate to 66 poles per day. In addition, experimental work is carried out in the factory to help in the development of such prestressed concrete products as beams, slabs, purlins, etc. An experimental table has been built where these products are made, tested, and then used for construction work at the factory. Particular attention is paid to the quality of concrete used in experimental work and it has been found that by suitably

modifying the grading, cement-aggregate ratios, and the water-cement ratio, concrete strengths up to 12,000 psi at 28 days can be achieved.

Pre-cast concrete jetty

Engineering (London), V. 185, No. 4792, Jan. 10, 1958, pp. 47-48

Reviewed by Aron L. Mirsky

Jetty, recently completed on the Thames, accommodates ocean-going colliers of 14,000 tons displacement. Foundations consist of 72 prestressed concrete cylinders, made of hollow precast units assembled vertically in a special berth built against the wharf wall and then prestressed. Superstructure of jetty was mainly cast in place and prestressed; soffits, stringers, plinths were, however, precast.

Prestressed concrete cylinders for two breakwaters

Robert N. Bruce, Jr., *Civil Engineering*, V. 29, No. 8, Aug. 1959, pp. 56-57

Describes the construction of two breakwaters using precast prestressed concrete cylinders. They were prestressed longitudinally by 12 post-tensioned bonded cables each consisting of 12 wires. Initial stress was 150,000 psi.

Information regarding size of piles, depth of penetration, fill material for space between piles, beam caps, lateral wave force, soil pressure, erosion effects, and other factors are briefly discussed.

Properties of Concrete

Strength of no-fines concretes

A. T. Tatevosian, *Beton i Zhelezobeton* (Moscow), June 1959, pp. 280-282

Reviewed by D. Watstein

Author's experimental study deals with the effect of W/C on the compressive strength of no-fines concretes having gravels of different specific surfaces. The thickness of cement paste coating on the gravel was determined for W/C ranging from 0.48 to 0.92; the corresponding coating thicknesses ranged from 0.9 to 0.1 mm.

Contrary to the established relation between W/C and strength for conventional concrete, an increase in the W/C of no-fines concrete within certain limits does not result in reduction of its strength. This is attributed to the fact that the more fluid cement paste collects at lower contact surface of individual particles where the increased amount of binder aids in resisting stresses. Thus, the strength can be increased by improving the mobility of the cement paste, which brings about the strengthening of the contacts at the expense of the cement coating elsewhere on the particles of gravel.

The range of sizes of particles has an important effect on the unit weight of no-fines concretes. For a ratio of largest to smallest particle of 1.5, the density of the concrete was 1.65; when this ratio was 5, the density of the concrete increased to 1.95. The compressive strength increased with density almost linearly from 20 to 40 kg per sq cm, for the density range cited above.

Design of curing cycles and calculation of compressive strength of concrete steam cured at different temperatures

E. J. Ershler, *Beton i Zhelezobeton* (Moscow), June 1959, pp. 262-266

Reviewed by D. Watstein

Author carried out a large number of tests on concrete cubes which were steam cured at temperatures of 40, 60, and 80 C. The proportions of concrete were 1:2:3 and the water-cement ratio was 0.45.

Author made use of J. M. Plowman's formula relating age and compressive strength of concrete, which is as follows:

$$R = A + B \lg \frac{N}{1000},$$

where R = compressive strength of concrete in psi, A and B are experimental constants, and N is the product of curing temperature in deg F (above 11 F) and the curing time in hours.

Author's data and several calculated examples show that Plowman's formula can be used with satisfactory precision to predict the compressive strength in a given curing cycle when the rate of heating of the medium does not exceed 20 C per hr. Greater precision can be attained in the use of this formula when the differences between the ambient temperature and the temperature of concrete are taken into account.

Resistance of concrete and mortar to chemical attack — Progress report on concrete corrosion studies, Part II

J. H. P. Van Aardt, *Bulletin*, No. 17, National Building Research Institute (Pretoria), Mar. 1959, pp. 1-22

Author's Summary

Data are presented to show how various curing conditions affect the corrosion resistance of hydraulic cement mortars. It is shown that carbonation has a beneficial effect on the sulfate resistance of portland cement mortars. Mortars made with an imported supersulfated slag cement were resistant to sodium sulfate, but were vulnerable to ammonium and magnesium sulfates. Calcareous aggregates can be used with advantage in acid and acid sulfate environments; moderate concentrations of chloride ions do not reduce expansion in sulfate solutions, and nitric acid solutions cause expansion of portland cement mortars.

If expansion takes place slowly, considerable dimensional change can be tolerated without a marked deterioration in the quality of the product, but a rapid rate of expansion causes a rapid drop in quality.

Pipe in concrete need not corrode

A. W. Peabody, *Consulting Engineer*, V. 12, No. 3, Mar. 1959, pp. 104-107

Reviewed by Aron L. Mirsky

Troubles in certain radiant heating installations involving steel pipe laid under concrete slabs have been traced to galvanic action involving pipe partly in vermiculite or earth and partly in concrete. The remedy is total encasement of the pipe in the concrete, easily accomplished by use of chairs or other supports; maximum aggregate size is also important, in terms of concrete cover of the pipe. Other possible remedies are shown to be less effective and/or more costly.

Structural Research

Hydraulic loading equipment for structural testing

Eivind Hognestad, *Journal*, Research and Development Laboratories, Portland Cement Association, Chicago, V. 1, No. 2, May 1959, pp. 30-37

Author's Summary

Major design features, performance characteristics, and applications are described for a system of hydraulic loading equipment involving inexpensive commercially available packed rams. By careful maintenance and use, an accuracy of load measurement within ± 1 to 2 percent can be obtained.

Properties of cast-in-place reinforced concrete constructions reinforced with precast prestressed elements

O. V. Mikhailov, *Beton i Zhelezobeton* (Moscow), May 1959, pp. 212-219

Reviewed by D. Watstein

Author points out that concrete in tension does not rupture instantaneously at the peak value of stress and that the stress-strain curve extends considerably beyond the value of strain of 0.015 percent normally assumed as the limit of concrete. However, this plastic range of concrete in tension cannot be utilized either in ordinary or prestressed concrete.

Tests of four rectangular beams reinforced with T-shaped prestressed elements show that nonprestressed concrete in such beams attains considerably greater limits than ordinary monolithic concrete. The tests showed that when the ratio of area of prestressed elements to the total cross section of beams varied from 0.10 to 0.14, the strain limit attained a value as high as 0.05 to 0.06 percent.

The largest width of crack observed in the nonprestressed concrete prior to formation of cracks in the prestressed elements was about 0.003 to 0.004 in.

The bond between the cast-in-place concrete and the precast prestressed elements was observed to be intact until the maximum load was reached. The manner of failure of these specimens was similar to that of conventional prestressed members.

Torsional vibrational experiments on rectangular concrete beams

R. Jones, *British Journal of Applied Physics* (London), V. 10, No. 1, 1959, pp. 51-52

Road Abstracts Apr. 1959

Earlier anomalous results obtained in measuring the dynamic shear modulus of concrete beams by a torsional resonance technique are explained as a result of experiments at the Road Research Laboratory, Harmondsworth. The experimental results are in close agreement with the theory proposed by Davies in 1938 which predicts discrepancies in the apparent shear modulus obtained at higher harmonic resonances due to warping of the rectangular section during vibration. It was found that the corrected shear modulus is not significantly influenced by the frequency at which it is measured. This conclusion agrees with the results obtained from torsional resonances on a concrete beam of circular cross section for which no correction is required.

Experiments on the compressive strength of anhydrite

W. J. Skinner, *The Engineer* (London), V. 207, No. 5377, Feb. 13, 1959, pp. 255-259; No. 5378, Feb. 20, 1959 pp. 288-292

Reviewed by Aron L. Mirsky

Interesting paper reporting application of experimental and statistical methods to a material which is not common in engineering, but which has characteristics which are shared by other, more common engineering materials, notably concrete, to which the theory and methods here described might profitably be applied.

Method of mining anhydrite is by the "pillar and stall" system, with an extraction ratio of 0.5. With a view to revising this empirical ratio and conserving anhydrite reserves, experiments were begun on the compressive strength of the material. Since it is brittle and shows decided size and shape effects (thus, compressive strength decreases as size increases), the theory of statistics of extreme values was resorted to. Paper gives a review of theories of brittle fracture, a description of the experimental procedure, a discussion of the effect of flaws, and a description and discussion of the application of the statistical theory to the experimental data and the calculation of the compressive strength of pillars of large (e.g., 20 ft height) size. The statistical theory itself is presented in an appendix.

Simplified method for the measurement of moment-influence field surfaces of plates (Vereinfachtes Verfahren zur Messung von Momenteneinflussflachen bei Platten)

W. Andra, F. Leonhardt, and R. Krieger, *Der Bauingenieur* (Berlin), V. 33, No. 11, Nov. 1958, pp. 407-414

Reviewed by Aron L. Mirsky

Describes apparatus for measuring offset distance to surface of distorted model from base plane established by three legs resting on surface of model. Apparatus uses standard electrical transducer and indicator, and is so arranged that measurements may be made in two orthogonal directions simultaneously, and combined to obtain influence surfaces for moment with effect of Poisson's ratio included. Models representing slabs of constant thickness are made of thin sheet metal (aluminum is recommended); slabs investigated may be of any shape (bridge decks, building floors, etc.) or thickness (which need not be constant, although this will affect the cost), or with any desired support conditions.

Measurements are utilized directly to obtain influence surfaces (in form of contour plats), or to obtain the principal moments and their directions. Results of several studies, notably of the Heilbronn Bleichinselbrucke (a skewed bridge deck curved in plan, with varying width) are given, as is a comparison between measured and calculated values.

In an addendum in the same issue, p. 443, Mehmel describes certain apparatus with a similar purpose, developed independently in his institute at Darmstadt.*

*For a full description of this apparatus, see Weigler, H., and Weise, H., "Modellstatisches Verfahren zur Aufnahme von Einflussflachen von Platten," *Beton- und Stahlbetonbau* (Berlin), V. 54, No. 5, May 1959, pp. 123-128.

Taxiway is a concrete laboratory

Engineering News-Record, V. 163, No. 8, Aug. 20, 1959, pp. 32-34

Describes upgrading of Biggs Air Force Base near El Paso, Tex. Taxiway T-3 is being built in three types of concrete pavement: conventional, 24-in. thick; reinforced, 19 in. thick; and prestressed, only 9 in. thick. The purpose of the taxiway work is threefold: to determine the relative economics of nonconventional pavements as opposed to that of conventional pavements; to indicate the conditions under which a nonconventional pavement might be more practical than conventional types; to check on the validity of certain design criteria and installation methods for nonconventional pavements being tested.

General

TV aids automatic batching

RICHARD S. HUHTA, *Concrete Products*, V. 62, No. 6, June 1959, pp. 32-35, 54

Description of a one-man automatic batching operation complete with closed-circuit television. The TV camera and photoelectric cells are used to help spot trucks under sand and gravel hoppers.

The long crossing

WILLIAM RATIGAN, William B. Eerdmans, Grand Rapids, Mich., 152 pp., $3.25

A narrative account of the life of D. B. Steinman up to the design and construction of the Mackinac Bridge between upper and lower Michigan. Entertaining and inspirational story of a great engineer and a major engineering achievement.

Epoxy plastics in architecture

GUY G. ROTHENSTEIN, *Progressive Architecture*, V. 40, No. 8, Aug. 1959, pp. 138-141

This article discusses epoxy, a comparatively young member of the plastics family. Briefly describes the development and make-up and then goes into properties of epoxies covering the phenomenal adhesion, dimensional stability, resistance to corrosion, toughness, and longevity. Applications as adhesives, coatings and primers, surfacing materials, and structural uses are touched on as well as method of application, cost, and possible expected future use. Compares and tabulates properties for unsupported epoxy films and epoxy surfacing compound (sand mixture).

Concrete sleepers

DONOVAN H. LEE, *Indian Concrete Journal* (Bombay), V. 33, No. 6, June 1959, pp. 194-198

The article, reprinted from *Railway Steel Topics*, discusses in a general way the development of flat bottomed rail track and the use of timber, steel, and concrete sleepers and the reasons why concrete sleepers are being increasingly used in Germany, France, and Britain. It also deals with experience with long welded rails and important advantages to be gained from the use of concrete sleepers with them.

Recent experience in three countries in connection with resilient pads under the rail and elastic fastenings is also mentioned. Practice and opinions in regard to prestressed concrete and reinforced pad type sleepers are discussed.

ACI NEWS LETTER

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'60 Nominees Announced

President--Joe W. Kelly

Vice-President--Raymond C. Reese

The ACI Committee on Nominations, headed by Thomas B. Kennedy, Waterways Experiment Station, Vicksburg, Miss., has announced candidates to be submitted to member ballot. Those elected from these nominees and any submitted, according to the Bylaws, by the membership at large, will take office at the 56th annual convention, March 14-17, at the Commodore Hotel, New York.

For President

Joe W. Kelly, professor of civil engineering, University of California, Berkeley, has been nominated to succeed Phil M. Ferguson as president for a 1-year term beginning at the February 1960 convention. He is now completing his second year as Institute vice-president.

An ACI member since 1926, Professor Kelly has actively participated in Institute affairs for over 30 years. He served on the Board of Direction from 1952-1954 and from 1956-1958 and has worked on a number of technical and administrative committees. He is currently a member of the Standards Committee and ACI Committee 611, Inspection of Concrete. He formerly served as author-chairman of Committee 611. During his tenure, the first three editions of the popular *ACI Manual of Concrete Inspection* were produced.

Well known in engineering circles as a concrete authority, Professor Kelly has authored or coauthored countless technical papers and articles on a wide range of subjects dealing

Joe W. Kelly

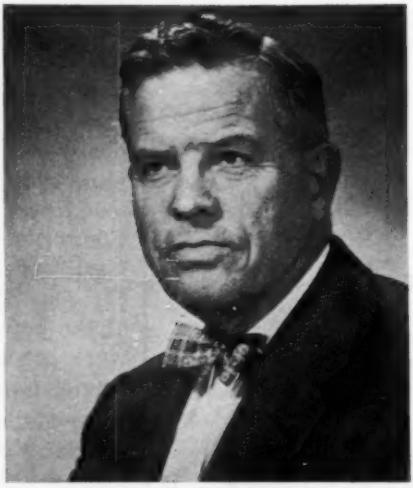
with concrete and concrete materials research. A frequent contributor of papers for JOURNAL publication, he was awarded the Wason medal in 1934 for the paper, "Cement Investigations for Boulder Dam with the Results Up to the Age of One Year," prepared in collaboration with Raymond E. Davis, R. W. Carlson, and G. E. Troxell. In 1946 he received the ACI Construction Practice Award with Bernerd D. Keatts for their paper, "Two Special Methods of Restoring and Strengthening Masonry Structures."

Professor Kelly graduated from Purdue University in 1921 and engaged in waterworks engineering for the next 2 years. Then, after a brief term as assistant in the testing materials laboratory at Purdue University, he joined the staff of the Portland Cement Association where he remained for 7 years. Returning to Purdue University as a concrete specialist of the engineering extension department, he prepared a practical booklet on concrete making for the small user of cement.

In May, 1931, he joined the staff of the Engineering Materials Laboratory at the University of California, where he engaged in the preparation of engineering reports and bulletins covering the many investigations of the laboratory. From his position as associate research engineer, he advanced to the rank of professor of civil engineering in 1939. Later he was named vice-chairman of the department, a position which he relinquished in November, 1958.

For Vice-President

Raymond C. Reese, consulting engineer, Toledo, Ohio, has been nominated for a 2-year term as vice-president. Joining ACI in 1936, he has participated in nearly every phase of Institute activity. Mr. Reese served for 8 years as chairman of ACI Committee 315, Detailing of Reinforced Concrete Structures, which is responsible for the *Manual of Standard Practice for Detailing Reinforced Concrete Structures.* He has been a member of the Standards Committee and is currently a member of the Board of Direction and chairman of ACI Committee 318, Standard Building Code.



Raymond C. Reese

An authoritative author in the field of reinforced concrete for many years, several of his papers have appeared in the JOURNAL. He received the Lindau Award in 1957 "in recognition of his years of effective coordination of practice and instruction through textbooks and technical papers on reinforced concrete."

Educated in engineering at Massachusetts Institute of Technology and Pierce Hall, Harvard, he spent several years as an industrial engineer with the General Electric Co. Since 1922 he has been in private practice as a consulting engineer; he has also been a consultant for the Hausman Steel Co. for over 30 years.

Mr. Reese has lectured in structural subjects at the University of Toledo and designed a number of rigid frame structures; he is considered a specialist in grain elevators and storage bins. He is author of the *CRSI Design Handbook*, and received the 1953 Award of the Concrete Reinforcing Steel Institute for "his contributions to reinforced concrete construction through design."

Lewis H. Tuthill, concrete engineer, Division of Design and Construction, California State Department of Water Resources, Sacramento, was elected to a 2-year term as vice-president at the 1959 convention and will continue in that office.

Board of Direction

Nominated to the four vacancies on the ACI Board of Direction for 3-year terms beginning at the February 1960 convention are: A. Allan Bates, Bruce E. Foster, Ben C. Gerwick, Jr., and James A. McCarthy.

A. Allan Bates, vice-president for research and development, Portland Cement Association, Chicago, is a scientist and chemical engineer with broad experience in industrial research.

Active in Institute affairs since 1946, Dr. Bates served on the Board of Direction from 1952-54. He assumed chairmanship of the ACI Building Committee in October, 1957, steering the committee through a successful campaign, culminating in the new ACI headquarters building.

Lewis H. Tuthill

Dr. Bates holds degrees from Ohio Wesleyan University and Case and a DSc from the University of Nancy in France.

Bruce E. Foster, assistant chief, Concreting Materials Section, National Bureau of Standards, Washington, D.C., has been an ACI member since 1948. He served on the Board of Direction from 1956-58 and is currently chairman of ACI Committee 212, Admixtures, and a member of the Standards Committee. Many of Dr. Foster's papers have been published in the JOURNAL.

Dr. Foster is a graduate of Colorado College, University of Oregon, and Stanford University. He joined the National Bureau of Standards in 1935 and most of his time has been spent in the California acceptance testing laboratories. He was associated with the testing of pozzolan cement for Bonneville Dam and was in charge of the Permanente branch laboratory which tested the cement for Shasta Dam and for most of the Pacific-area construction during World War II.

Ben C. Gerwick, Jr. president of Ben C. Gerwick, Inc., San Francisco, is a former president of the Prestressed Concrete Institute.

Mr. Gerwick, graduate of the University of California, joined Ben C. Gerwick, Inc. in 1940 for a brief period prior to active military duty. He rejoined the organization in 1946 and since that time has filled positions of field engineer, office engineer, vice-president, and in 1952 became president of the company.

Mr. Gerwick, a member of ACI since 1956, has been active in the Northern

A. Allan Bates

Bruce C. Foster

Ben C. Gerwick, Jr.

James A. McCarthy

California Chapter of the Institute since its formation.

James A. McCarthy, professor of civil engineering, University of Notre Dame, joined ACI in 1940 and has been engaged in technical committee work for a number of years. Professor McCarthy is currently a member of ACI Committee 115, Research; and a member of the Standards Committee. He was a member of the Technical Activities Committee from 1955-1956.

Professor McCarthy is chairman of the Indiana State Board of Registration for professional engineers and land surveyors.

Nominating Committee

Members of the Nominating Committee in addition to Chairman Kennedy are Arthur R. Anderson, Anderson, Birkeland and Anderson, Tacoma, Wash.; Frank Kerekes, Michigan College of Mines and Technology, Houghton; Douglas McHenry, Portland Cement Association, Chicago; T. C. Powers, Portland Cement Association, Chicago; Walter H. Price, U. S. Bureau of Reclamation, Denver; C. P. Siess, University of Illinois, Urbana; and Anton Tedesko, Roberts and Schaefer Co., New York, N.Y.

Twenty candidates have been chosen for the 1960 Nominating Committee, five of whom will be elected at the March convention. Candidates are:

J. O. Ackerman
F. H. Bienhauer
D. L. Bloem
W. A. Cordon
Ernest Gruenwald
W. C. Hansen
Samuel Hobbs
Eivind Hognestad
J. R. Janney
M. S. Ketchum, Jr.
R. W. Kluge
R. C. Mielenz
Eric Molke
G. M. Nordby
H. F. Peckworth
D. H. Pletta
Emil Schmid
E. W. Thorson
Robert P. Witt
K. B. Woods

56th Annual

ACI Convention

March 14-17 * Commodore Hotel * New York City

Prestressed Girder Tests

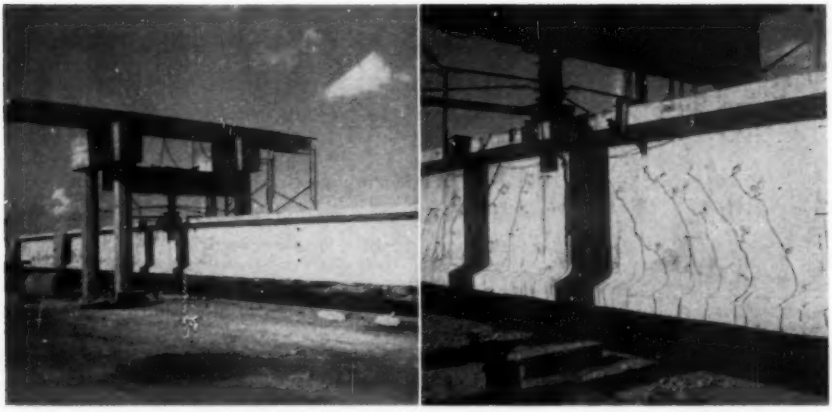
DURING THE PLANNING of the reinforced concrete IATCS/Center Building for the Civil Aeronautics Administration, Honolulu, Hawaii, the possibilities of prestressing were investigated. A 90-ft prestressed girder was designed and tested.

At midspan the girder was 4 ft 6 in. high with the top of the girder tapering down to 4 ft high at each end. The top and bottom flanges were 18 in. wide and about 5 in. high; central web was 5 in. thick. The design called for a concrete strength of 6000 psi at 28 days. Thirty-two $\frac{7}{16}$-in. diameter seven-wire strands were used in groups of eight in the post-tensioned girder.

For the test a single concentrated load was applied at the center line of the girder by a 100-ton hydraulic jack. The concentrated load was selected to produce the same maximum bending moment as the corresponding uniform load it represented. The maximum test load specified by the owners was (DL+ 3 LL), producing a total bending moment of 2850 ft-kips including 435 ft-kips from the DL of the girder. The design moment under full DL + LL was 1500 ft-kips.

The test girder performed satisfactorily, and exhibited the typical characteristics of prestressed beams — strength, stiffness, and toughness. The cracks at the maximum load were numerous and closely spaced, indicating that the grout injected into the cable ducts performed satisfactorily. The midspan deflection at DL + 3 LL was $6\frac{3}{4}$ in., and the residual deflection was a little over $\frac{1}{2}$ in.

The girder was designed by Park & Yee, Ltd., Consulting Engineers, Honolulu, and tested under their supervision at the casting yard of Concrete Engineering Co., Ltd. Aluminum cone anchorage devices developed by Concrete Technology Corp., Tacoma, Wash., were used.



Post-test view of girder (left). Crack pattern (right) after DL + 3 LL

OLD DAN'L NEVER KNEW...

This newly completed concrete highway bearing the name of Daniel Boone speeds traffic through the very countryside old Dan'l once traveled and knew only as a tangled wilderness.

The section of the Daniel Boone Expressway shown above was built by Millstone Construction, Inc., St. Louis.

Modern construction genius employing Laclede's complete designed reinforcement service is helping create a multi-million dollar national system of expressways, of which the Daniel Boone is a part. To withstand the constant pounding of cars to and from ever-expanding suburbs, these new freeways must be built *strong*.

Much of the needed strength comes from Laclede designed reinforcing steels—multi-rib reinforcing bars, welded wire fabric, welded dowel spacers, center and recess joints, tie bars and accessories.

LACLEDE

LACLEDE STEEL COMPANY

SAINT LOUIS, MISSOURI ◆ Producers of Steel for Industry and Construction

Positions and Projects

56th annual convention New York City, March 14-17

ACI convention-goers attending the 56th annual meeting of the Institute at the Commodore Hotel, New York City, March 14-17, will hear a technical program geared to the interests of the designer, builder, research man, and others in fields related to the concrete industry.

The final technical program announced by the Technical Activities Committee includes approximately 20 papers to be presented at five sessions by eminent concrete authorities.

The Institute's Board of Direction and Technical Activities Committee will begin their work ahead of the general sessions with meetings on Saturday and Sunday, March 12 and 13.

Monday, March 14 and Tuesday morning, March 16, have been designated "committee days" and numerous ACI technical committees will convene in open session to review current work and aims.

The first general session will be called to order on Tuesday, March 15, at 2 p.m. The session will feature proposed ACI Standards and committee reports.

Concurrent sessions on March 16 will be devoted to model tests and highways in the morning, and design and structural research in the afternoon. On March 17 Committee 201, Durability of Concrete in Service, Hubert Woods, Portland Cement Association, chairman, will present a symposium on restoration of deteriorated concrete. A concurrent session on design and construction will also be held Thursday morning.

Committee 115 will conduct its annual research forum on Thursday afternoon, March 17, bringing latest reports of tests on concrete in progress at institutions throughout the world.

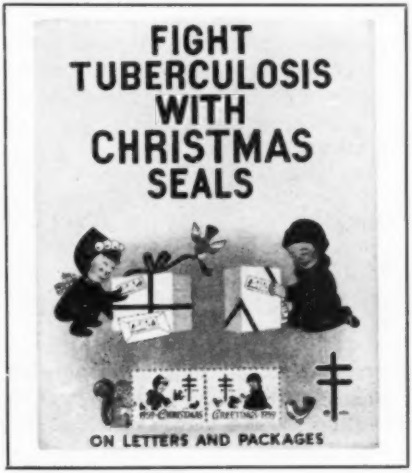
Annual awards will be presented by President Phil M. Ferguson at the Awards Luncheon, Wednesday, March 16.

Roger H. Corbetta, president of the Corbetta Construction Co., New York City, is general chairman of the local committee which is arranging a number of "extras" to make the 56th annual meeting thoroughly enjoyable as well as worthwhile.

ACI Southern California Chapter launches fall series

The Southern California Chapter launched its fall series with a dinner meeting on September 23 at the Rodger Young Auditorium, Los Angeles, featuring Richard R. Bradshaw as guest speaker.

Mr. Bradshaw, consulting structural engineer, Van Nuys, spoke on "Noteworthy Buildings in Foreign Lands." In recent months he has traveled extensively in many foreign countries and has an outstanding collection of photographs covering the range of ancient to modern in building construction. He also touched on his recent visit to Moscow as consultant on the struc-



tural safety of the United States Exhibit building.

Mr. Bradshaw, a member of the Southern California Chapter, is currently serving on two ACI technical committees: Committee 328, Limit Design—Joint ACI-ASCE; and Committee 334, Concrete Shell Structures.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete

A. Litvin
Armour Research Foundation of Illinois Institute of Technology
Chicago, Ill.

Committee 318, Standard Building Code

J. L. Peterson
J. L. Peterson, Inc.
Long Beach, Calif.

Morris Schupack
Schupack and Zollman
New York, N.Y.

Committee 505, Design and Construction of Reinforced Concrete Chimneys

William G. Saville
Embasco Services, Inc.
New York, N.Y.

Committee 616, Coating for Concrete

Bruce R. Schoenfeld
Engineering Consultant
Ross, Calif.

Committee 623, Cellular Concretes

Richard E. Barnes
Perlite Institute
New York, N.Y.

Committee 717, Practice in Low-Pressure Steam Curing

Vance E. Swift, Jr.
Concrete Materials, Inc.
Charlotte, N.C.

Expanded Shale Clay and Slate Institute makes grant to NBS

The Expanded Shale Clay and Slate Institute has made a grant to the National Bureau of Standards, Washington, D.C., for a comprehensive study of creep and shrinkage of expanded shale lightweight concretes. This study will be conducted over a period of 5 to 10 years on concretes made from aggregates produced by members of ESCSI.

The initial stage of the program is designed to determine shrinkage, creep and other properties of steam cured concretes loaded in 24 hr. Additional stages will study the effect of variables such as age at loading, strength of concrete, intensity of load, shape of section, type of cements, and types of curing. This study will be conducted on concretes made with 20 or more expanded shales and several representative heavyweight aggregates.

The program will be conducted under the direction of D. E. Parsons, chief, Building Technology Division, by David Watstein, chief, Structural Engineering Section, and Thomas W. Reichard, physicist, Structural Engineering Section.

Chastain and Tindel form new consulting firm

T. Z. Chastain and T. A. Tindel have formed Chastain and Tindel, Inc., new Atlanta consulting structural engineering concern, in a merger of two firms.

Mr. Chastain since 1954 has operated a company bearing his name. I. E. Morris, board chairman of Morris and Tindel, will serve as structural consultant and board chairman of the new organization.

High-Lift Bridge Piers

MASSIVE CONCRETE PIERS in the substructure for the $22 million international bridge near Ogdensburg, N. Y., were designed in cross section to facilitate single lift placement.

Work on the $6 million substructure for the bridge was completed by Merritt-Chapman & Scott Corp., New York, more than 9 months ahead of schedule. The substructure included five land piers and 28 water piers.

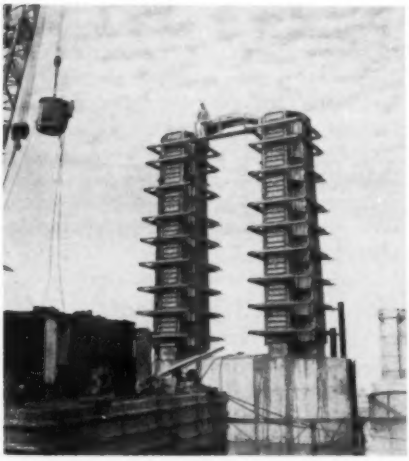
Unusual feature of the forms, designed and fabricated by Blaw-Knox Co., used to construct the concrete piers was that they supported placements as high as 41 ft without external bracing or guys, and helped to minimize finish work. Instead of tie rods, heavy structural members were used in the form to resist concrete pressure in the unusually high single lifts.

Pier foundations ranged in depth from 22 to 66 ft below mean low water and contained as much as 14,000 cu yd of concrete. A total of 70,000 cu yd of concrete was required for all substructure work. One station of pier columns, varying in height from 13 to 41 ft, arch tie, and pier cap contained an average of 45 cu yd of concrete.

The contractor installed a batch plant at Prescott on the Canadian side of the river. Concrete was transported in 3- and 4-cu yd buckets by tug-pushed barges. Barge-mounted cranes equipped with 96 ft long booms lifted the buckets.

Column forms were capped as a unit over a 1 ft high stub, cast with the pier foundation, and positioned with leveling jacks.

Column forms tapered from top to bottom according to the height of the



In spite of large pier dimensions (left) no tie rods were used to concrete these pier columns or arch tie for one section of the international bridge between Canada and the United States, near Ogdensburg, N. Y. The arch tie and pier cap (right) were stripped as a single unit and repositioned for another placement in less than 3 hr

pier. For example, the form units employed in casting the 41 ft high pier columns measured 8½ x 12 ft at the bottom and tapered to 8⅛ x 8⅓ ft just below the pier cap. These pier column forms were designed to withstand concrete pressure in excess of 2000 psf.

Concrete was placed at a rate of 1000 cu ft per hr and cured 24 hr. Stripping and repositioning the column forms consumed only 8 hr. Bolts were loosened, the bond broken, and the 20-ton unit was shifted to a barge. Bolts were retightened, structural members positioned, and the unit was ready for another column placement.

Rustication strips — utilized mostly for pier design purposes — remained in the pier after the form unit was moved. The strips were later pried loose and repositioned with forms at the next column site.

In addition to these pier column forms, Blaw-Knox also provided steel forms for arch ties and pier caps. Cross-sectional dimensions for arch ties varied from 12 to 12½ ft, clear span, and 3⅔ to 4⅔ ft. The 8½ ft thick pier cap was octagonal in shape at the top of the two column piers. Concrete for both these sections cured 6 days.

Support brackets for arch tie forms were set in column piers as they were cast. Holes were used to locate template bolts which became anchor bolts for the supporting brackets. Side and end panels of pier cap forms were fitted over a 5 ft thick stub cast with the pier column. The forms rested on screw jacks used for alignment and leveling. The single unit arch tie and two pier cap form sections were lifted in less than 3 hr.

The 1 mile long span over the St. Lawrence River links Chimney Point, N. Y., just east of Ogdensburg, with Johnstown, Ontario, Canada. Main bridge spans will total ½ mile in length. These will carry a two-lane, 26 ft wide roadway 125 ft above the main St. Lawrence River channel. The span will serve as a principal link in the Penn-Canada Expressway, now under construction.

Structural conference held at University of Colorado

About 200 engineers and architects from Colorado and parts of Wyoming attended a structural engineering conference at the University of Colorado, Boulder, on Friday, November 13. Prestressed concrete formed the basis for the conference which was sponsored by the university's civil engineering department and the Portland Cement Association.

T. Y. Lin, civil engineering professor, University of California, Berkeley, was one of the prominent authorities addressing the conference.

Other speakers during the day-long meeting included W. Burr Bennett, Structural Bureau, PCA, Chicago; James M. Hastings of the Denver firm Ketchum, Konkel and Hastings; and Donald G. Leitch, instructor on concrete research at the University of Colorado.

James Chinn, associate professor of civil engineering, acted as conference chairman.

Horn elected president of Buchart Engineering

Russell E. Horn, formerly executive vice-president of Buchart Engineering Corp., York, Pa., has been named president. In his new position he will direct and coordinate all engineering activities of the company's operations in highway, bridge, and structural work. Previously he was with the Corps of Engineers and is a Colonel in its reserve force.

POZZOLITH . . . makes good concrete better

ALLEN STEAM PLANT project at Belmont, N.C. is part of Duke Power Company's continuing expansion program to meet growing power needs in the Piedmont area. Project includes approximately 73,000 cubic yards of POZZOLITH concrete.

POZZOLITH concrete specified again for Duke Power Company's continuing $200 million expansion

Duke Power Company—with the largest privately-owned, steam-operated electric capacity in the southeast—handles its own design, engineering, construction and concrete production. These operations are directed by C. T. Wanzer, Chief Engineer —C. E. Watkins, Construction Manager and R. L. Dick, Resident Engineer.

On the Allen Steam Plant project—several concrete mixes were designed with POZZOLITH and Duke Power Company's fly ash. The mix that best met job requirements most economically was:

Cement— 353 lbs.
Fly Ash (Duke Power)— 117 lbs.
Sand—1215 lbs.
1" Crushed Stone—2057 lbs.
Total Water— 30 gal.
POZZOLITH

The job concrete was easily pumped more than 600 feet through an 8-inch line from the automatic batching plant. Slump of 2" to 3" was maintained—there was excellent workability and cohesiveness . . . and high, uniform strength . . . 2250 psi at 7 days . . . 3700 psi at 28 days . . . and 4900 psi at 90 days. The specified 28-day strength of 3000 psi was easily exceeded.

The local Master Builders field man will welcome discussing your requirements. Remember, POZZOLITH concrete is higher in quality and more economical than plain concrete or concrete made with any other admixture.

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***POZZOLITH is a registered trademark of The Master Builders Company for its concrete admixture to reduce water and control entrainment of air and rate of hardening.**

Reinforced Concrete Pipe Chosen For

This 96" diameter reinforced concrete pipe easily met the rigid requirements of ASTM Specification C76-57T. Tests indicated this reinforced concrete pipe was capable of loads exceeding 100,000 pounds, demonstrating the strength and durability.

S. J. Groves and Sons Company, Minneapolis, Minnesota—General Contractor

Abbott Contractors, Chicago, Illinois—Sub-contractors

Waukesha Cement Tile Company, Milwaukee, Wisconsin—Pipe Suppliers

American Vitrified Products Company, Milwaukee, Wisconsin—Pipe Suppliers

Lock-Joint Pipe Company, South Beloit, Illinois—Pipe Suppliers

Richard I. Bong Air Force Base Runway Drainage

More than 48,000 linear feet of concrete pipe was included in the Corps of Engineers specifications for the Richard I. Bong Air Force Base at Kansasville, Wisconsin. Many sizes were required, the largest being 96" inside diameter. All reinforced concrete pipe was manufactured and pre-tested to meet the rigid requirements of ASTM Specification C76-57T. The 96" pipe was required to meet a loading test of 96,000 pounds. It met this requirement easily and could have withstood loads exceeding 100,000 pounds!

Reinforced concrete pipe is designed in conformance with specifications comprehending the laws of mechanics plus the results of thousands of tests; it is manufactured by men experienced with concrete product production; and it is pre-tested to demonstrate its strength. The pre-testing is done by means of the 3-edge bearing test which is the most severe loading to which the pipe will ever be subjected!

Reinforced concrete pipe installations, in service for many years, are testimony to the strength, durability and watertightness of concrete pipe reinforced with Welded Wire Fabric.

Concrete pipe manufacturers insist on top-quality reinforcing, meeting rigid specifications – that's why so many of them use American Welded Wire Fabric. This quality product, with its machine-made accuracy, assures the proper distribution of steel. It is pre-fabricated from cold-drawn, 60,000 psi yield strength wires. All intersections are electrically welded to assure mechanical anchorage in the concrete. Wire diameters as large as 0.505" on 2", 3" or 4" centers can be furnished. Its rigid specifications assure superior reinforcement.

Write American Steel & Wire, Dept. 9315, 614 Superior Avenue, N. W., Cleveland 13, Ohio for complete technical literature.

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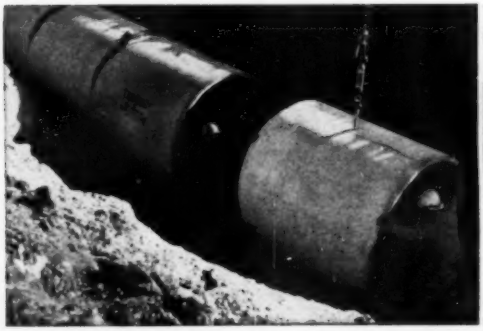
United States Steel Export Company, Distributors Abroad



Sections of concrete pipe go in place more quickly and with no job interruptions. Its uniform size insures tight joints and proper alignment.

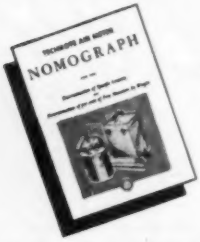
USS American Welded Wire Fabric, with its small diameter, closely spaced steel members, distributes load stresses evenly throughout the concrete pipe. It is the ideal concrete pipe reinforcement, and can be easily formed to the proper size cage diameters.

American Welded Wire Fabric

TECHKOTE AIR METER..

FIRST CHOICE AROUND THE WORLD

The Techkote Air Meter now provides a multi-range testing instrument that vastly increases the usefulness of the air meter. It is now possible to measure accurately—within a fraction of a degree—entrained air with one instrument on numerous materials such as lightweight concrete, mortar, plaster and soil, in addition to regular concrete.



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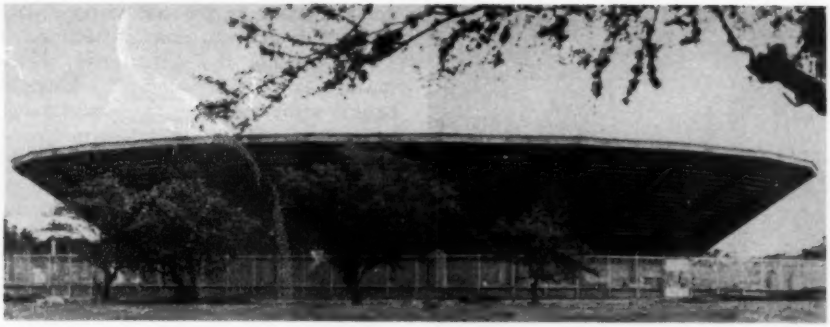
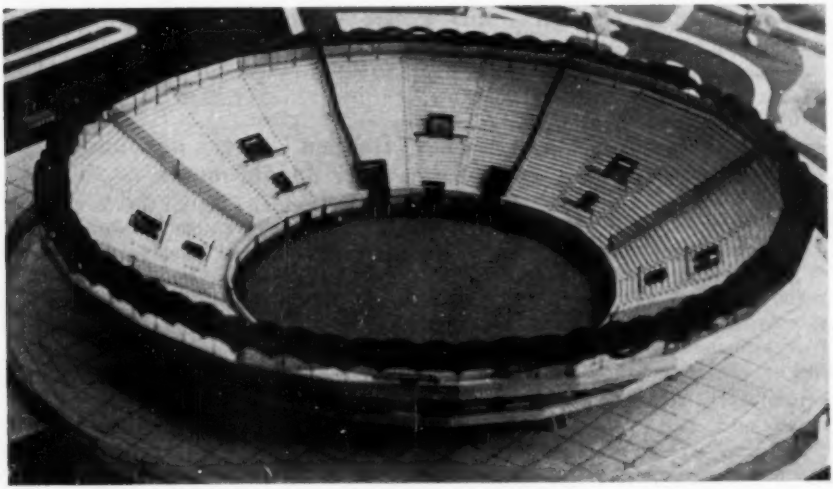
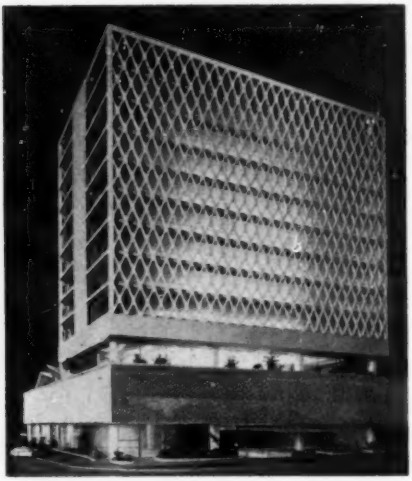


Prestressed Concrete Bull Ring

THE SAUCER SHAPED Plaza de Toros at Cali, Colombia, has an exterior diameter of 312 ft and rises 48 ft above ground level. Accommodating 18,000 spectators, there are 15 entrances to the ring — seven leading directly from the main gates and eight from a suspended corridor hanging from the structure. The reinforced concrete stadium was prestressed by 34 circular exterior cables, each with six to ten high tensile steel wires of 5 mm diameter. The $2\frac{1}{2}$ in. thick treads and 4 in. thick risers span between 24 cantilever ribs which rest on two foundation rings and cantilever up and out 60 ft from the base wall. The concrete had a strength of 4500 psi at 28 days and a concrete densifier was added for better workability. Most of the formwork was plywood to obtain a pleasing architectural surface.

The structure was designed by Julian Guerrero and Jaime Camacho; Guillermo Gonzalez and Carlos Hernandez were the structural engineers. The prestressed design was by Domenico Parma. The structure was built by Gino Faccio and Francisco J. Villaquiran, Civil Engineers.





Structural "Sculpture" in 13-Story R/C Building

CONSTRUCTION HAS STARTED on the new 13-story home office building in Los Angeles of the American Cement Corp. and its largest division, Riverside Cement Co. The 240,000 sq ft building represents, novel, practical, and artistic solutions in concrete. Emphasis has been placed on the proportion and shape of the structure, achieving a simplified silhouette with a unique sculptural quality.

Subterranean, second, and third levels have been allocated to parking. Retail stores and shops and a conference room for the building's tenants will be adjacent to the restaurant on the ground floor. The ten floors of the tower will be devoted to offices.

A "transition platform" permits use of a different column spacing in garage area and a different structural system below the transitional platform from that used in the office areas above the garage areas.

Reinforced concrete bearing walls on north and south elevations will be sculptured screen designed and engineered to support the full lateral and vertical loads of the ten tower floors. They also provide sun protection.

The core of the ten stories of the towers is an inverted T. Elevator banks, utility services, and mechanical equipment will be in the core.

A basic engineering feature of the 13-story structure is the use of beams and only eight supporting columns under the tower portion. Broad footings extending below the subterranean parking level will distribute the weight of the tower.

Reinforced concrete screens will surround the second and third parking levels eliminating the use of mechanical ventilation and appreciably decreasing construction costs.

The spacious lobby characterized by textured reinforced concrete walls, sculptured concrete lighting fixtures with accents of stained glass, and reflecting pools illuminated by free-form sculptured concrete lighting standards, are among other unusual uses of concrete.

The roof of the executive offices of Riverside Cement Co. on the fourth floor will be a folded concrete plate of unique geometric design.

The building was designed by the architectural firm of Daniel, Mann, Johnson and Mendenhall; Peter Kiewit Sons' Co. are the contractors. Owners are Arthur Gilbert and Edward Rothschild, industrial and commercial developers.

Dundee Cement announces staff appointments

The Dundee Cement Co., in the process of erecting a new $25 million plant at Dundee, Mich., have announced several new staff appointments.

Gerald J. Mulvey assumes the post of manager of the Chicago distribution station and Lewis A. Zorn heads up the Monroe distribution station at Monroe, Mich.

New appointments to the Chicago district sales force include John E. Shreve and David E. Walker.

Alpha appoints Bonsteel district sales manager

Austin Bonsteel has been appointed New York district sales manager by Alpha Portland Cement Co., Easton, Pa.

He succeeds J. D. Bell, vice-president, who has assumed new duties as Eastern Division sales manager.

Wirkus appointed to PCA post

Eugene A. Wirkus has been appointed field engineer on the metropolitan Los Angeles staff of the Portland Cement Association.

Mr. Wirkus was formerly an associate highway engineer in the design department of the California Division of Highways.

Barschdorf joins Harris staff

Milton P. Barschdorf, Colonel, Corps of Engineers (ret.), has joined Frederic R. Harris, Inc., consulting engineers, as its Lower Mississippi Valley area representative. He will make his headquarters in New Orleans, La.

Colonel Barschdorf recently retired from 21 years of duty with the Corps of Engineers, his last assignment being district engineer of the Vicksburg, Miss. district.

Permanente awards contract for construction of Hawaiian plant

Permanente Cement Co. has awarded a contract for construction of initial facilities at its $13,500,000 Hawaiian cement plant to Anderson-Westfall Co. of Portland, Ore. Located on the island of Oahu the plant will have an annual productive capacity of 1,700,000 barrels of cement. It will utilize a nearby land-based coral deposit as the principal source of raw material.

Permanente also announced that fabrication of the shell of the 453 ft long rotating kiln for the Hawaiian facility is well underway at the Napa, Calif., plant of Kaiser Steel's Fabricating Di-

The first book on foundations to emphasize economy in designing and building—

FOUNDATION DESIGN AND PRACTICE

AN ECONOMIC VIEW

by J. H. Thornley

Written especially to help the designing engineer choose the correct foundation for a given structure at the least possible cost to the owner (without in any way reducing the safety of the structure) — this book should prove indispensable to engineers concerned with rising costs and increased competition.

Every phase of the subject is discussed, from selection of site and consideration of the demands of the superstructure to an analysis of contracts and building codes. Various types of foundations are covered in full (with the help of 49 detailed line drawings), and a special six-page Elimination Table enables the designing engineer to narrow his field by excluding those types which will not suit a particular job.

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CLINTON Welded Wire Fabric

A steel backbone for concrete structures

The giant steelman symbolizes the years of experience, top quality materials and rigid quality controls that enable CF&I to produce dependable steel products for all industries. Contractors rely on one of these —CF&I-Clinton Welded Wire Fabric—to add the tensile strength of steel to the durability of concrete.

CF&I-Clinton Welded Wire Fabric is ideal for reinforcing *all* types of concrete—both light and heavy construction. The wire mesh adds years of trouble-free life to the concrete because it helps cushion the shock of heavy loads, distributing the stress over a wide area. The steel reinforcing also minimizes cracking during the setting period because it provides anchorage for the concrete. And the steel fabric holds the concrete tightly together to prevent heaving due to extreme temperature changes.

CF&I-Clinton Welded Wire Fabric is easy to use. The fabric unrolls smoothly and lays flat . . . it can be shaped and cut quickly. It's available in a wide range of gages and meshes for every reinforcing requirement. CF&I-Clinton Welded Wire Fabric is also furnished in mat form to meet individual specifications.

For full details—and prompt delivery—call the nearby CF&I sales office.

FREE! Send for new 32-page catalog, "CF&I Steel Products for the Construction Industry".

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vision. The giant kiln will be shipped in 15 sections to Honolulu.

Contracts for erection of the kiln and other construction phases will be let through Henry J. Kaiser Construction Co. as the project develops.

Chiu made associate professor

Arthur Chiu, assistant professor in the Department of Civil Engineering at the University of Hawaii in Honolulu, has been made an associate professor.

New Spancrete plant for Green Bay area

West Allis Concrete Products Co., Wisconsin firm headed by Henry Nagy and Arthur Hintz, opened its new plant at Valders, Wis., early in July. The first big contract is for 500,000 sq ft of Spancrete roof slabs for a building project. When that job is completed the facility will serve the Green Bay area with 800,000 sq ft of roof slabs a year. Another Spancrete plant is planned for the Minneapolis area.

Allen made vice-president of Hamilton Kent organization

Hamilton Kent Manufacturing Co., Kent, Ohio, has announced the appointment of H. S. Allen to the post of vice-president.

Mr. Allen received his civil engineering degree from Oklahoma State College, and has since worked in various engineering capacities both in America and abroad. For many years he was associated with the Lock Joint Pipe Co., Kansas City, Mo., where he served as district manager. He has been an ACI member since 1927.

Alpha plant wins three safety awards

Employees at Alpha Portland Cement Co., Jamesville, N. Y. plant, received three safety awards at presentation exercises held at the plant on August 12.

Two awards, from the Portland Cement Association, were the Safety Trophy award for an accident-free year (1958) and the 1000 Day Club award, presented for a record of over 1000 consecutive days with no lost-time injuries. The third award was a Certificate of Commendation from the National Safety Council for operating over 400,000 manhours without a disabling injury.

NCMA moves to Washington

Headquarters of the National Concrete Masonry Association are now located in Washington, D. C. Formerly in Chicago, the association moved to its new location in August.

Benham appointed to Oklahoma engineering registration board

Dave Benham, senior partner in the Benham Engineering Co., Oklahoma City and Muskogee, Okla., has been appointed by Gov. J. Howard Edmondson to the five-member Oklahoma Board of Engineering Registrations. His appointment is for a 5-year term.

LOOKING AHEAD

Jan. 25-29, 1960—Stress Measurement Symposium, Arizona State University, Tempe, Ariz.

Feb. 1-5, 1960—Committee Week, American Society for Testing Materials, Hotel Sherman, Chicago, Ill.

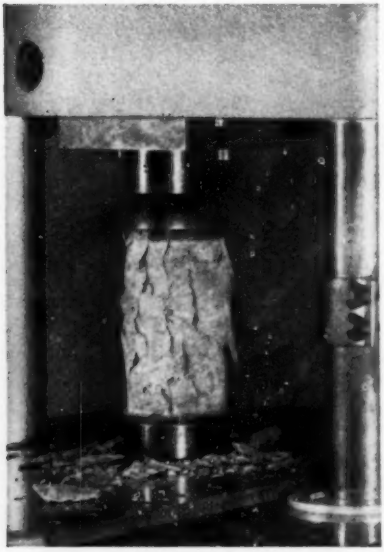
Feb. 15-19, 1960—Annual Convention and Exposition, National Sand and Gravel Association, Conrad Hilton Hotel and Coliseum, Chicago, Ill.

Feb. 22-24, 1960—Annual Convention and Exposition, National Crushed Stone Association, Conrad Hilton Hotel, Chicago, Ill.

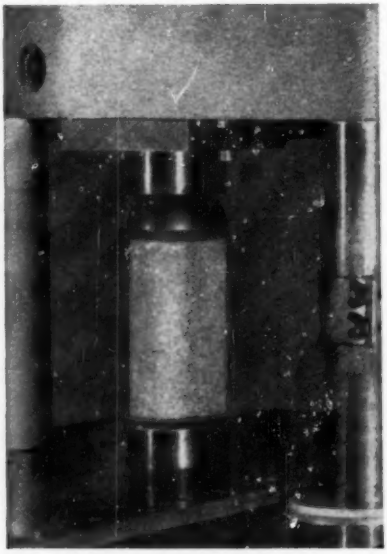
Mar. 14-17, 1960—56th Annual Meeting, American Concrete Institute, Commodore Hotel, New York, N. Y.

Jan. 18-21, 1960—58th Annual Convention, American Road Builders Association, Netherland Hilton Hotel, Cincinnati, Ohio.

Jan. 18-21, 1960—Annual Convention, National Limestone Institute, Statler-Hilton Hotel, Washington, D.C.



3100 psi Plain Concrete



3850 psi Placewel Concrete

No extra cost—for today's biggest increase in concrete quality

Use Johns-Manville liquid Placewel . . . less than 3 fluid oz./sk. of cement

TEST DEMONSTRATES why Johns-Manville Placewel® has been used in over 5,000,000 cu. yds. of concrete. With this liquid admixture, it is possible to use a minimum paste content . . . and get maximum concrete quality.

With Placewel, you use the minimum amount of paste needed to lubricate the mix and bind the aggregate particles together in the hardened state.

By eliminating excess paste you reduce costs, minimize drying shrinkage and make concrete less susceptible to attack by aggressive solutions and weathering.

A Placewel engineer is available on request for technical assistance. Write today for his address. Johns-Manville, Box 14, New York 16, New York.

Less than 3 fluid oz. of Placewel per sack of cement will—

- Increase strength up to 30%
- Improve quality at no extra cost
- Increase workability and placeability
- Reduce bleeding and segregation
- Reduce construction, maintenance costs
- Give controlled air-entrainment

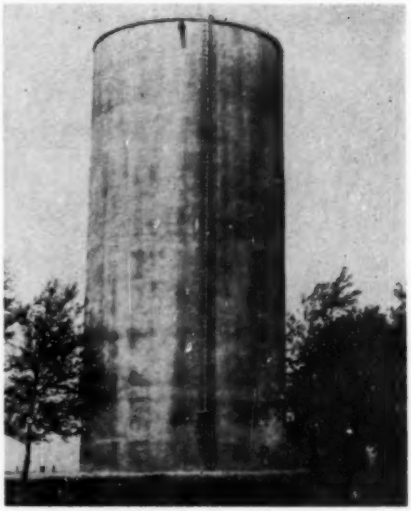
JOHNS-MANVILLE

JOHNS MANVILLE JM PRODUCTS

LOW MAINTENANCE COSTS played an important part in the selection of prestressed concrete for two 660,000-gal. stand pipes in Wadsworth, Ohio. Although the steel tanks cost $84,270 as against $141,300 for prestressed concrete, it was estimated that over a 50-year period the steel tanks would cost about $316,000 to maintain whereas for all practical purposes the maintenance cost for concrete was negligible.

One stand pipe measures 36 ft in diameter by 91 ft high and the other measures 40 ft in diameter by 72 ft high. The stand pipes, constructed by Preload Concrete Structures, Inc., Westbury, N. Y., are of all pneumatic-mortar construction except for the monolithic 8 to 10 ft deep foundations. Walls are tapered from 8 in. thick at the base to 5 in. thick at the top. The stand pipes are roofed with 2 in. thick domes.

In erecting the wall, Preload constructed a cylindrical wood form and shot pneumatic mortar against it to build up the desired thicknesses. Walls contain nominal vertical steel reinforcing for



temperature stress during erection and the roof is reinforced with a wire mesh. The tanks were wrapped with prestressing wire by a self-propelled wire-winding machine. The wire was applied in three layers with shotcrete between layers and a final coating of ¾ in. thick mortar.

Byce establishes engineering firm

Richard C. Byce announces the establishment of the firm of Richard C. Byce and Associates, Inc. with offices in Kalamazoo, Mich. Mr. Byce will serve as president of the new firm.

Rahel Shalon elected president of RILEM

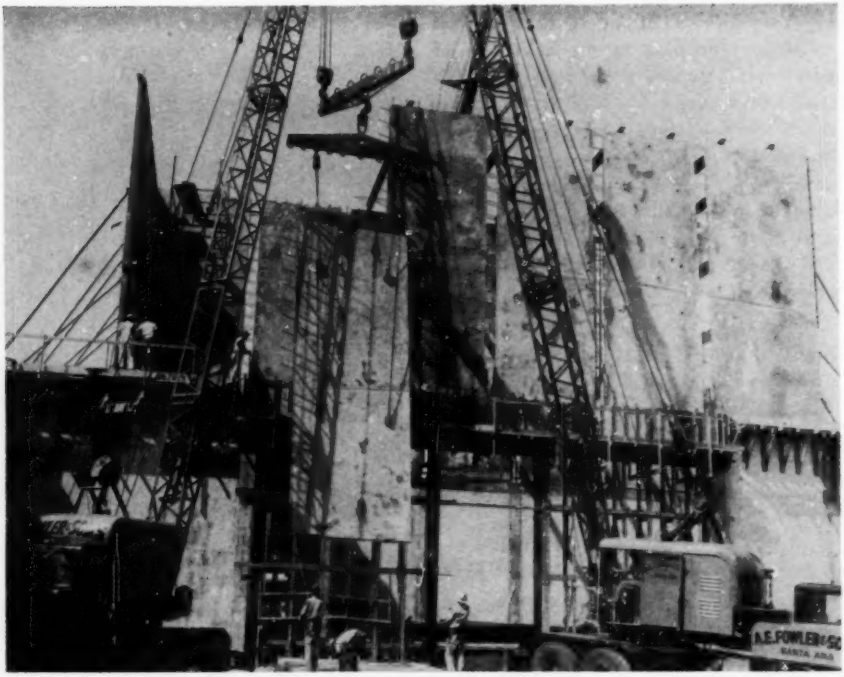
Rahel Shalon, chief of the Building Research Station at the Technion-Israel Institute of Technology, Haifa, was elected president of RILEM, the International Union of Testing and Research Laboratories for Materials and Structures, at the annual meeting in Belgrade, Yugoslavia.

Professor Shalon was graduated from the Technion-Israel Institute of Technology in 1930. She has published numerous papers dealing with construction materials and is considered one of the world's foremost authorities on reinforced concrete. Professor Shalon was coauthor of the paper on "Influence of Sea Water on Corrosion of Reinforcement" published in the June, 1959, JOURNAL.

Demarest appointed to Alpha sales force

James C. Demarest has joined Alpha Portland Cement Co. as a sales representative in northern New Jersey.

Before joining Alpha he had been affiliated with F. J. Brotherton and the Shoreland Construction Co.



LARGE LIGHTWEIGHT CONCRETE PANELS are lifted into position 60 ft high on the Del Rio Senior High School, Santa Ana, Calif. Expanded shale aggregate was used in the concrete for the tilt-up panels and the roof slab. The 8 in. thick tilt-up panels were 36 ft high and 14 ft wide. The use of lightweight concrete reduced the dead load of each panel to 17 tons or 50 lb per cu ft thus enabling the cranes to place the panels with an extension boom.

By using lightweight concrete in the roof deck over the auditorium, it was possible to reduce the seismic forces concentrated overhead at any given point by one third.

Harold Gimeno was architect and engineer was Floyd Weaver. General contractor on the project was Means and Ulrich; the cement contractors were Nordstrom and Faccau; the Rocklite lightweight aggregate concrete was supplied by Western Concrete and Equipment Co. Twining Laboratories was responsible for all testing and inspection. All of the participating firms are located in Santa Ana.

Balke expands engineering firm

Harry Balke, consulting engineer, Cincinnati, Ohio, announces that his firm, Harry Balke Engineers, has moved into new offices with expanded facilities in Cincinnati.

Ideal stockholders approve acquisition of Volunteer

Ideal Cement Co. stockholders have approved acquisition of the Volunteer Portland Cement Co. of Knoxville, Tenn. Combined production capacity will be about 37,000,000 bbl annually.

56th ANNUAL ACI *Convention*

8-Sessions-8

- Standards & Committee Reports
- Model Tests
- Highways
- Design & Structural Research
- Materials
- Restoration of Deteriorated Concrete
- Design & Construction
- Research Session

March 14-17, 1959

Let's make this the "biggest" convention ever. Plan now to attend these "fact-filled" sessions and assure yourself this opportunity to gain the latest and most up-to-date information available on concrete and concrete construction. Make your reservations early.

Hotel Commodore

New York City

Bonato accepts position with Betonma International

Emil Bonato formerly assistant manager of Betonma North America, St. Catharines, Canada, has accepted a position in the American office of Betonma International located in Niagara Falls, Canada. Firm headquarters are in Antwerp, Belgium.

Hydraulic group honors Straub

Lorenz G. Straub, director of the St. Anthony Falls Hydraulic laboratory and head of the University of Minnesota civil engineering department, was elected an honorary member of the International Association for Hydraulic Research at its convention last week in Montreal, Canada.

Mr. Straub is the first honorary member to be selected from the Western Hemisphere and only the fifth honorary member in the 23-year history of the association. Previously elected to honorary membership were engineers from Sweden, Switzerland, Russia, and Italy. Mr. Straub served as president of the International Hydraulic association from 1948 to 1955.

Ambric opens materials testing laboratories

The Ambric Testing and Engineering Associates, Inc., announce the openening of their new testing laboratories in Philadelphia. Physical tests and chemical analyses of structural and mechanical materials, including concrete, masonry materials, and soils, can be made.

The three top men in the new organization, all consultants in this field for many years, are Harold M. Carson, president; Donald D. Meisel, chief engineer; and Lewis G. Bricker, chief chemist.

POUR CONCRETE ALL WINTER?

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To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. All who have an interest in concrete are eligible for membership. The grades of membership are described below.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI Journal provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings, they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

ACI's world-wide membership is growing in extent and participation —traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

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P. O. Box 4754, Redford Station
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The undersigned hereby applies for admission to the American Concrete Institute as ☐ Individual ☐ Corporation ☐ Contributing ☐ Junior ☐ Student Member and agrees to be governed by the Charter and Bylaws.

Name and complete mail address of proposed membership (Address to which Journal is to be mailed—please letter) _____

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For our records, please complete both sides of application.

NOTES on Membership Classification

The ACI advertising and editorial departments need definite information concerning job title, classification, business affiliation, and principal responsibility of all members. Thus, the information requested on the classification form below is especially important.

The applicant should designate his title or position such as: construction superintendent, research director, technician, draftsman, structural engineer, inspector, plant superintendent, highway engineer. The occupation of the applicant should be that which is **most frequently** performed. The principal responsibility of the applicant is that specific area of his job for which he is **primarily concerned.**

Classification is not based on what interests you, but on what you do—what office or job you fill. When completing the form below, please check the one item in each section that is most applicable to that particular section.

ACI editors want to know your interests—they welcome suggestions as to what you'd like to see discussed in the ACI Journal and of the possible sources. Please attach a separate note or letter.

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Do you ☐ Specify ☐ Authorize ☐ Recommend, purchase of materials or equipment?

The ACI Membership Directory will be sent—as available—only on request. Check here if you wish to receive the latest edition. ☐

Stone and Webster appoint Casselman to key position

Theodore E. Casselman, Jr., formerly assistant engineering manager in the Boston office of Stone and Webster Engineering Corp., has been appointed manager of the New York office.

Mr. Casselman joined Stone and Webster in 1940 as a project engineer and has worked on the design of many chemical and petrochemical plants built by Stone and Webster in the United States and various parts of the world. He was promoted to chief engineer of the Chemical Division in 1956 and made assistant engineering manager in 1958.

Errata

The following correction should be made in the review of "Prestressed sheet piles enclose jetties," on p. 184 in the "Current Reviews" section of the August, 1959, JOURNAL.

The article originally appeared in the *Engineering News-Record*, V. 162, No. 19, May 1959, pp. 59-60,63 and not in *Civil Engineering*, V. 29, No. 5.

* * *

In "General Elastic Analysis of Flat Slabs and Plates," by John F. Brotchie, published in the August, 1959 JOURNAL, the following corrections should be made.

p. 135 — in the second sentence of the second paragraph, change "parameters" to "parameter s."

p. 146 — in the first paragraph, third line below the equation for M_c, change "$\frac{1}{4}K_c$" to "$1/(4K_c)$."

* * *

The following corrections should be made in the discussion of "Role of Cement in the Creep of Mortar" in the Part 2 September, 1959, JOURNAL (*Proceedings* V. 55).

p. 1558 — the first sentence of the second paragraph should read, in part, "Tensile strength, σ_r, of concrete rings subjected to restrained shrinkage, δ, was determined; . . ."

p. 1559 — in the next to last line, ε_o should read $\bar{\varepsilon}_o$.

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Emmons on retirement furlough

Walter J. Emmons, professor of highway engineering and assistant dean and secretary of the College of Engineering at the University of Michigan, is retiring after 32 years service. For the first year he will be on "retirement furlough." Retirement furlough marks the end of regular classroom work and is considered a year of consultantship to the university.

Closner appointed Preload President

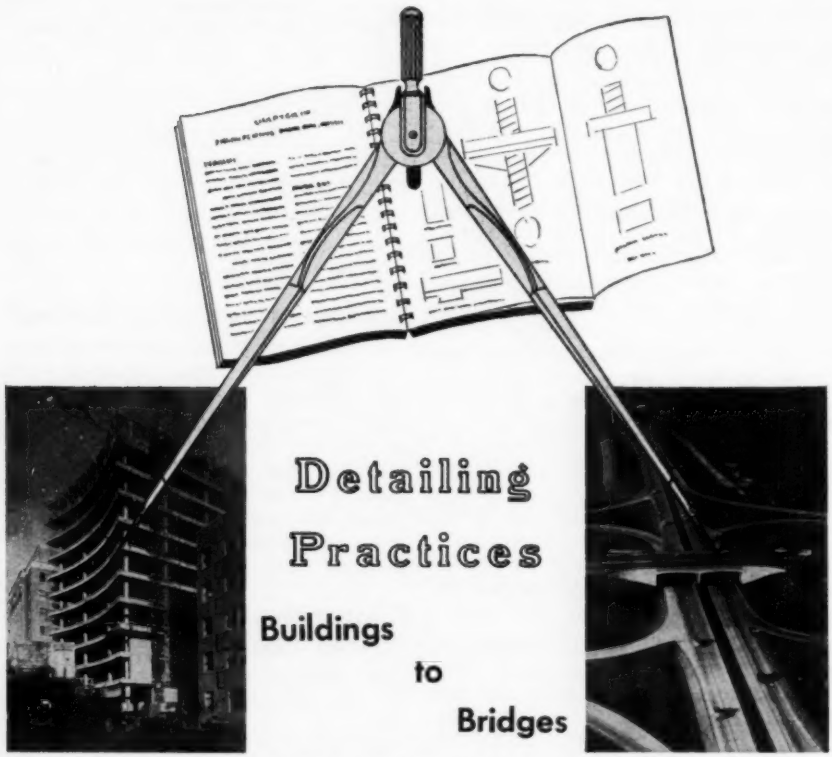
The Preload Co., Inc. has announced the appointment of J. J. Closner as president.

Mr. Closner has been with the Preload organization since 1949 serving in various capacities including southwest district manager, assistant general manager, and vice-president in charge of sales and operations.

American-Marietta acquires Perry Jones facilities

All quarries and stone crushing facilities of Perry Jones, Carbondale, Kan., have been purchased by American-Marietta Co.

Fred E. Bellamy, president of Concrete Materials and Construction Division, American-Marietta Co. of Cedar Rapids, Iowa, announced that the firm will be combined with the division and the name changed accordingly. Present personnel of the Perry Jones firm will be retained and the company contemplates no immediate changes in the operation.



58 Illustrative Drawings

The ***Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-57)*** incorporates under one cover the previous separate editions on detailing of buildings and highway structures. It is a correlation of the latest improved methods and standards for preparing drawings for the fabrication and placing of reinforcing steel. Sections on detailing and fabricating shop practice are translated into practical examples in typical drawings. Spiral bound to lie flat, it is the only publication of its kind in English and is meeting wide acclaim among designers, draftsmen, and engineering schools.

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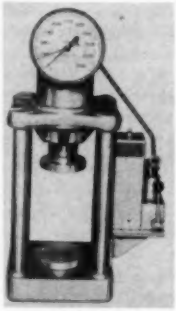
Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Portable concrete testing machine

Soiltest announces a new 250,000 lb capacity portable concrete testing machine, Model CT-710 for on-the-site testing.

It is designed for testing standard 6 x 12-in. cylinders and also for testing beams, 2-in. cubes, concrete blocks and other concrete specimens, when proper attachments are used. Drain tile, brick, plastic, ceramics, and similar materials can be tested in compression or in flexure.



The lightweight tester is 38 in. high and 21 in. wide including the pump assembly. Entirely self contained and no pressure or electrical connections are required for operation. —Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

Form clamp

Rocform Corp. is manufacturing the Adapt-A-Clamp form hardware. Made of steel channels, the clamps can be fastened with a single hammer blow. Fastening the clamps locks adjoining panels and aligns them. The clamp eliminates wood walers. Designed for use with Rockform tie-rods, they can be adapted to other types. — Rocform Corp., 15160 W. Eight Mile Rd., Detroit 15, Mich.

Portable space heater

The 1960 model of the Champ-Heater operates on a principle entirely new to the space heater field according to the manufacturer. The principle, similar to that used in residential type oil burners, involves two entirely separate sources of air; one to support combustion, the other (on top of the heater) for heater air circulation.

Styled in a new low silhouette, the Champ-Heater Model 120-T operates on regular 110-115 v, 60 cycle ac, and uses kerosene or fuel oil. Available in three sizes; thermostatically controlled to prevent overheating.—Champion Heater Co., 3700 Forest Park, St. Louis 8, Mo.

Splice chucks

Supreme Products, makers of Supreme Strand Chucks for prestressers, is now offering a Splice Chuck in 6 sizes for ⅛, 7/16, and ½ in. strand. The purpose of this chuck is to connect ends of strand and hold them under all operating tensions. Catalog No. 349 available.—Supreme Products Corp., 2222 South Calumet Ave., Chicago, Ill.

Wetting agent

Deynor Corp. is marketing Multiwet No. 50, a concentrated liquid wetting agent which is added to the dry mix material. Manufacturer claims that 15 percent less water is required for the same slump. The agent entrains about 3½ percent air. Brochure available.—Deynor Corp., 1 Depot Plaza, Mamaroneck, N. Y.

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Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

CALCIUM CHLORIDE IN CONCRETE — 40-page technical manual contains data of major effects of calcium chloride, early and ultimate strength, cold weather protection, high-early-strength cement, and air-entrained concrete. Charts illustrate laboratory data from nationally recognized technical organizations.—Calcium Chloride Institute, 909 Ring Building, Washington 6, D. C.

AIR ENTRAINMENT METER—Soiltest offers 6-page bulletin which describes a precision type of air entrainment meter used in testing and designing concrete mixes. Contains operating instructions and information on maintenance and recalibration of the unit.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

THE WELDING OF CONCRETE REINFORCING BARS—Booklet contains joint details and recommended procedures for welding reinforcing bars. Tables show kinds and grades of reinforcing bars and schedule of preferred joint details for arc-welded splices in reinforcing bars.—Bethlehem Steel Co., Bethlehem, Pa.

ACI Records System Revised

Effective Oct. 1, 1959, ACI membership records were converted to the IBM punched card system of record maintenance. This decision by the Executive Committee was prompted to assure speedier and more efficient handling of membership records at less expense.

Under the new system, dues invoices, and JOURNAL and other large mailings will be handled mechanically in a fraction of the time now required.

Should your JOURNAL fail to reach you, or is misaddressed, due to the records changeover, please notify ACI headquarters so prompt correction can be made.

HOW TO MAKE OPENINGS IN FLEXICORE DECKS—This booklet sums up the things you want to know about openings in Flexicore decks.—Flexicore Co., Inc., 1932 East Monument Ave., Dayton 1, Ohio.

LACLEDE 7-WIRE STRAND FOR PRESTRESSED CONCRETE—Completely illustrated brochure shows step-by-step procedures by which Laclede 7-Wire Strand for pretensioning is manufactured, beginning in the open hearth furnace and ending in the finished product.—Laclede Steel Co., The Arcade Bldg., St. Louis, Mo.

MONOLITHIC FLOORING MATERIALS (Bulletin 3-4)—New bulletin on REZKLAD flooring materials. Installation can be made directly to the concrete slab or, in the case of an old building, can be applied on the existing floors to up-grade the surface.—Atlas Mineral Products Co., Mertztown, Pa.

CSI CATALOG (No. 59A)—Catalog contains illustrations and brief descriptions of 73 testers or equipment manufactured as standard products by Custom Scientific Instruments, Inc. CSI also build testers or equipment of all types for individual and general needs.—Custom Scientific Instruments, Inc., Kearny, New Jersey.

SOLVING TUNNEL CONCRETING PROBLEMS (MBR-P8)—How excavating and concreting problems encountered on 12 major tunneling projects were solved is the subject of this 28-page Master Builders Co. publication. Highway, railway, and water tunnels and their construction techniques are illustrated by photos and accompanying texts.—The Master Builders Co., Cleveland 3, Ohio.

MATERIALS EVALUATION FACILITIES AND SERVICES (Bulletin 5902) — Bulletin describes the United States Testing Co. Materials Evaluation Division's facilities and services for analysis, development, research, and inspection of materials and products.—United States Testing Co., 1415 Park Ave., Hoboken, N. J.

EMERI-EPOX SURFACING PRODUCT (Bulletin EX-1)—Two-page bulletin sets forth the properties, recommended applications, and special advantages of Emeri-Epox, a floor surfacing and patching material combining emery and epoxy resin. Choice of available colors is listed.—Walter Maguire Co., Inc., 60 East 42nd St., New York 17, N. Y.

MASONRY ANCHORING HANDBOOK—Revised edition, 48-page pocket size catalog. Prepared especially for architects, engineers, and users of masonry anchors and drills.—The Rawlplug Co., Inc., New Rochelle, N.Y.

CONCRETE COLOR CARD—Wall chart color card featuring eight large Dust-On color chips illustrating many consumer uses for Dust-On colored concrete. Floated and troweled on top of fresh concrete it becomes an integral part of the surface.—Tamms Industries Co., 228 North LaSalle St., Chicago 1, Ill.

ENGLISH ABSTRACTS OF RUSSIAN TECHNICAL JOURNALS — Listing of some 100 Soviet technical periodicals abstracted regularly by U. S. Government agencies and released to the public through OTS. The new listing supersedes one published in August 1958. Available without charge. — Office of Technical Services, U. S. Department of Commerce, Washington 25, D. C.

THE FIRST STEP IN CONSTRUCTION — A 16 mm color and sound motion picture showing several famous construction jobs. Included are scenes of the excavation work at the Niagara Power Project, the channel deepening on the St. Lawrence Seaway, and the tunnel work for the new Fort Pitt tunnels in Pittsburgh. The 15-min film is suitable for showing to engineering and construction groups; city, state, and federal highway personnel; and student groups. No charge.—Joy Manufacturing Co., Henry W. Oliver Bldg., Pittsburgh 22, Pa.

POZZOLITH EMPLOYED IN BRIDGE CONCRETE (Report No. 11)—Special bridge concreting problems and how they were solved is the theme of this 16-page Master Builders Co. publication. Case history reports of more than 200 bridges are covered. Includes discussion of hot and cold weather concreting, the use of lightweight aggregate in bridge work, and placing and finishing problems encountered.—The Master Builders Co., Cleveland 3, Ohio

STANDARD CALCIUM CHLORIDE SOLUTION — Pamphlet explains how to prepare standard calcium chloride solution. Prepared especially for ready-mixed concrete products, it contains illustrated recommendations for procedure in making solution, and a chart on recommended amounts of solution per batch.—Calcium Chloride Institute, 909 Ring Bldg., Washington 6, D. C.

EMERY FACT FOLDERS—A series of four Fact Folders as a convenient means of collecting information on industrial weighing systems. Folders cover the basic art of weighing, accuracy in weighing systems, the Emery "rolling ball" head, and the instrumentation used with modern weighing systems.—The A. H. Emery Co., New Canaan, Conn.

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